GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT

North Corner Riverside Drive and Le Harve
Avenue
Lake Elsinore, California
for
Lakeshore Pointe, LLC

September 18, 2014

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Lakeshore Pointe, LLC
One Better World Circle
Temecula, California 92590-3712

Attention: Mr. Allen Nuñez

Project No.: **14G178-1**

Subject: **Geotechnical Investigation**
Proposed Residential Development
North Corner Riverside Drive and Le Harve Avenue
Lake Elsinore, California

Reference: Geotechnical Investigation and Liquefaction Evaluation, Proposed Multi-Family Residential Development, Riverside Drive SW of Eisenhower Drive, Lake Elsinore, California, prepared for Classic Pacific, prepared by Southern California Geotechnical, Inc. (SCG), SCG Project No. 05G289-1, dated December 8, 2005.

Gentlemen:

In accordance with your request, we have conducted a geotechnical investigation and liquefaction evaluation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

Site Preparation

- Initial site preparation should include stripping of any surficial vegetation from the site. At the time of our investigation, ground surface cover throughout most of the site consisted of sparse to moderate grass and weed growth with scattered trees, bushes, and stumps in the northern portion of the site.
- The near surface soils consist of native alluvium with moderate porosity, and relatively low strengths.
- Remedial grading is recommended to be performed within the new building pad areas in order to provide a uniform layer of compacted structural fill soils beneath the proposed building pads. The existing soils within the building pad areas should be overexcavated to a depth of 5 feet below existing grade and to a depth of 5 feet below proposed pad grade, whichever is greater.
- Overexcavation in the proposed interior street areas may be limited to a depth of 2 feet below existing grade, or 2 feet below proposed grade, whichever is greater.
- After overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The overexcavation subgrade should be moisture conditioned, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

Liquefaction

- Our site-specific liquefaction evaluation indicates that some of the on-site soils are subject to liquefaction during the design seismic event.
- The liquefaction analysis indicates a potential for total dynamic settlements of 1.6 to 2.6 inches at the site. The liquefaction-induced differential settlements within the building areas are expected to be on the order of 1.3 inches. Assuming that these settlements occur across a distance of 100± feet, a maximum angular distortion of 0.0011± inches per inch would result.
- Standard practice dictates that the proposed improvements can be supported on shallow foundation systems, with the understanding that some cosmetic distress could occur due to liquefaction. Such distress will be typical of buildings of this type, in this area, in the event of a large earthquake.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,000 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings, due to the presence of potentially liquefiable soils. Additional reinforcement may be necessary for structural considerations.

Building and Garage Floor Slabs

- Conventional Slabs-on-Grade, 5-inches minimum thickness due to liquefaction potential.
- Minimum reinforcement of the floor slab should consist of No. 4 bars at 18-inches on center in both directions, due to the presence of potentially liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer.

Driveways and Exterior Flatwork

- Conventional Slabs-on-Grade, 4-inches minimum thickness due to liquefaction potential.
- Minimum reinforcement consisting of conventional welded wire mesh (6x6-W1.4xW1.4 WWF). The actual floor slab reinforcement should be determined by the structural engineer.

Pavements

ASPHALT PAVEMENTS (R = 30)		
Materials	**Thickness (inches)**	
	Interior Cul-de-sacs (TI = 4.5)	Interior Collector Streets (TI = 5.5)
Asphalt Concrete	3	3½
Aggregate Base	5	7
Compacted Subgrade (90% minimum compaction)	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS		
Materials	**Thickness (inches)**	
	Interior Cul-de-sacs (TI = 4.5)	Interior Collector Streets (TI = 5.5)
PCC	5	5½
Compacted Subgrade (95% minimum compaction)	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 14P322R, dated August 5, 2014. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of the subject site, this investigation also included a site-specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the north corner of Riverside Drive and Le Harve Avenue in Lake Elsinore, California. The site is bounded to the northwest by a single family residential subdivision, to the northeast by retail buildings and a single family residential subdivision, to the southeast by Riverside Drive, and to the southwest by Le Harve Avenue. The general location of the site is illustrated on the Site Location Map, included as Plate 1 in Appendix A of this report.

The subject site consists of an L-shaped property, 13.28± acres in size. The site comprises four (4) rectangular-shaped parcels which are presently vacant and undeveloped. An unimproved dirt road trending roughly northwest-southeast traverses the subject site from Riverside Drive to the northwesterly adjacent residential neighborhood. A small walnut grove is present in the north corner of the site. The ground surface cover consists of exposed soil with moderate native grass and weed growth over the majority of the site and exposed soil with sparse native grass and weed growth in the walnut grove area.

Detailed topographic information was not available at the time of this report. Based on visual observations, site topography slopes gently downward to the southeast at an estimated gradient of approximately 2 percent, toward Lake Elsinore. An ascending slope is located along the northwest property line. The slope is approximately 3 feet in height and has an inclination of approximately 3h:1v. With the exception of the aforementioned slope, there was estimated to be 25 feet of elevation differential across the subject site.

3.2 Proposed Development

A site plan prepared by MAA Architects was provided to our office by the client. The site plan indicates that the northern one-third of the site will be developed with forty-seven (47) single family residences and the southern two-thirds of the site will be developed with nine (9) multi-family residential buildings, a clubhouse building, and a swimming pool. The single family residential lots will be approximately 3,400 ft^2 in size. The multi-family residential buildings will consist of 14 to 20 unit apartments. The area surrounding the multi-family residential structures will be developed with asphaltic concrete pavements for parking and drive lanes.

It is assumed that proposed buildings will be of wood frame and stucco construction, presumably supported on conventional shallow foundations and concrete slab on grade floors. Based on the proposed construction, maximum column and wall loads are expected to be on the order of 30 kips and 2 to 3 kips per linear foot, respectively.

The proposed development is not expected to include any significant amounts of below grade construction such as basements or crawl spaces. Based on the assumed topography, cuts and fills of up to 5 to 6± feet are expected to be necessary to achieve the proposed building pad grades.

3.3 Previous Studies

SCG performed the referenced geotechnical investigation for the subject site in 2005. At that time, the proposed site development consisted of twenty (20) two-story multi-family residential buildings. This report was developed using the 1997 Uniform Building Code (UBC) and 2001 California Building Code (CBC). As part of the geotechnical investigation, six (6) borings were drilled to depths of 20 to 50± feet.

The borings encountered topsoil/root mat material at all of the boring locations to a depth of 2± inches. Native alluvial soils were encountered beneath the topsoil at all of the boring locations extending to the maximum depth of 50± feet. The native soils consisted of loose to medium dense silty fine sands and fine sandy silts to a depth of 30± feet and medium dense to very dense fine to coarse sands to a depth of 50± feet.

The referenced geotechnical report included a site specific liquefaction evaluation. The results of the liquefaction evaluation indicated that potentially liquefiable soils were identified at depths of 32 to 37± feet and 47 to 50± feet, under the design seismic event that was specified by the 1997 UBC/2001 CBC. The results of the liquefaction evaluation indicated potential total and differential dynamic settlements of 1 and ½ inches, respectively.

Geotechnical design considerations identified during the referenced investigation include loose, settlement prone soils in the upper 5± feet below the existing site grades, and liquefaction potential. Remedial grading was recommended in order to remove the loose soils and to provide a uniform layer of compacted structural fill beneath new floor slabs and foundations. Structural mitigation was recommended for the potential liquefaction settlements. It was recommended that foundations and floor slabs be designed to resist the potential total and differential settlements calculated for the liquefaction evaluation.

An addendum to the referenced report, dated December 20, 2005, presented the results of soluble sulfate testing. The results of this testing indicated soluble sulfate concentrations ranging from 0.001 to 0.28 percent at two of the boring locations, indicating that the on-site soils possess negligible to severe sulfate concentrations based on current ACI 318 guidelines.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of three (3) borings advanced to depths of 50 to 55± feet below currently existing site grades, as part of the liquefaction evaluation. All of the borings were logged during drilling by a member of our staff. As discussed in Section 3.3 of this report, the SCG previously performed six (6) borings at the subject site, extending to depths of 20 to 50± feet as a part of the referenced investigation.

The borings were advanced with hollow-stem augers, by a truck-mounted drilling rig. Representative bulk and in-situ soil samples were taken during drilling. In-situ samples were taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. The samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content.

The approximate locations of the borings are indicated on the Boring Location Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B. The boring logs for the six borings performed for the referenced geotechnical investigation are included in Appendix G.

4.2 Geotechnical Conditions

Alluvium

Native alluvium was encountered at the ground surface at all of the boring locations which extend to the maximum depth explored of 55± feet. The alluvium exposed at the ground surface possesses a slightly disturbed appearance, as if the site had been disced or tilled recently. The near-surface native alluvial soils consist of loose to medium dense fine sandy silts and silty fine sands with occasional stiff silty clay strata extending to depths of 18 to 22± feet below the existing site grades. Beneath these soils the native alluvium consists of interbedded sands, silty sands, silty clays and clayey silts.

Groundwater

Free water was encountered during the drilling of Boring No. B-7 at a depth of 37± feet. Free water was not encountered during the drilling of Boring Nos. B-8 and B-9. Additional measurements were performed within the open boreholes after the hollow stem augers were withdrawn. However, due to caving within the open boreholes, it was not possible to record any further groundwater readings. Based on the initial water level measurements taken during drilling, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth of 37± feet in the southern portion of the site and at depths of greater than 51½± feet in the central and northern portions of the site, at the time of the subsurface exploration.

As part of our research, we reviewed available historic groundwater data in order to determine a historic high groundwater table for the subject site. Our research included the California Water Data Library website, the Western Municipal Water District Cooperative Well Measuring Program quarterly reports, and environmental reports available from the Geotracker data base. However, the available data was not sufficient to determine a historic high groundwater level for the subject site, due to the lack of data available for wells located near the site and at similar elevations. The historic high groundwater level was therefore determined by an examination of the samples recovered during drilling at the site. Many of the samples recovered at depths greater than 18± feet possess a mottled appearance and possess iron oxide staining, indicating that anaerobic bacteria may have been present in these samples during saturation. Samples recovered from depths less than 18± feet did not possess iron oxide staining. Therefore, the historic high groundwater level for the site is considered to be about 18± feet.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. The field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

Moisture Content

The moisture contents for all of the recovered samples are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Soluble Sulfates

Representative samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	Soluble Sulfates (%)	ACI Classification
B-7 @ 0 to 5 feet	0.130	Moderate
B-9 @ 0 to 5 feet	0.120	Moderate

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829 as required by the California Building Code (CBC). The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

Sample Identification	Expansion Index	Expansive Potential
B-8 @ 0 to 5 feet	0	Very Low (Non-Expansive)

Grain Size Analysis

Limited grain size analyses have been performed on several selected samples, in accordance with ASTM D-1140. These samples were washed over a #200 sieve to determine the percentage of fine-grained material in each sample, which is defined as the material which passes the #200 sieve. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these tests are presented on the Boring Logs.

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on selected samples. This test is used to determine the Liquid Limit and Plastic Limit of the soil. The Plasticity Index (PI) is the difference between the two limits. The results of the Atterberg Limits testing are used to evaluate the liquefaction potential of the fine grained soils encountered below the historic high ground water table. The results of the Atterberg Limits testing are presented on the Boring Logs.

Consolidation

At the time of the referenced geotechnical report, selected soil samples were tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-12 in Appendix G of this report.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample was tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil type or soil mixes may be necessary at a later date. The result of the testing is plotted on Plate C-1 in Appendix C of this report.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations. The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structures should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Furthermore, SCG did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the site is considered to be low.

The potential for other geologic hazards such as seismically induced settlement, lateral spreading, tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low.

Seismic Design Parameters

The 2013 California Building Code (CBC) was adopted by all municipalities within Southern California on January 1, 2014. The CBC provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

The 2013 CBC Seismic Design Parameters have been generated using U.S. Seismic Design Maps, a web-based software application developed by the United States Geological Survey. This software application, available at the USGS web site, calculates seismic design parameters in accordance with the 2013 CBC, utilizing a database of deterministic site accelerations at 0.01 degree intervals. The table below is a compilation of the data provided by the USGS application. A copy of the output generated from this program is included as Plate E-1 in Appendix E of this report. A copy of the Design Response Spectrum, as generated by the USGS application is also included in Appendix E. Based on this output, the following parameters may be utilized for the subject site:

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	2.333
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.940
Site Class	---	F*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	2.333
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.410
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.556
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.940

*The 2013 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site coefficients are to be determined in accordance with Section 11.4.7 of ASCE 7-10. However, Section 20.3.1 of ASCE 7-10 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structure is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structure has a fundamental period greater than 0.5 seconds, a site specific seismic hazards analysis would be required and additional subsurface exploration would be necessary.

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration that is consistent with maximum considered earthquake ground motions, as required by the 2013 CBC. The peak ground acceleration (PGA_M) was determined in accordance with Section 11.8.3 of ASCE 7-10. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-10. The web-based software application U.S. Seismic Design Maps (described in the previous section) was used to determine PGA_M, which is equal to 0.931g. A portion of the program output is included as Plate E-2 in Appendix E of this report.

Liquefaction

Review of the Riverside County GIS website indicates that the subject site is located within a mapped zone of high to very high liquefaction susceptibility. Therefore, the scope of this investigation included a detailed liquefaction evaluation in order to determine the site-specific liquefaction potential.

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is determined as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable. Additionally, in accordance with Special Publication 117A, clayey soils which do not meet the criteria for liquefiable soils defined by Bray and Sancio (2006), loose soils with a plasticity index (PI) less than 12 and moisture content greater than 85% of the liquid limit, are not considered to be susceptible to liquefaction. However, soils with a PI between 12 and 18 may be moderately susceptible to liquefaction if the moisture content is greater than 80 percent of the liquid limit. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable.

The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction analysis was performed for the three (3) 50±-foot deep boring locations. The liquefaction potential was analyzed at the boring locations utilizing a PGA_M of 0.931g related to a 6.96 magnitude seismic event. A historic high groundwater depth of 18 feet was used in the analysis, as discussed in Section 4.2 of this report.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined using the equation for volumetric strain due to post-cyclic reconsolidation (Yoshimine et. al, 2006). This procedure uses an empirical relationship between the induced cyclic shear strain and the corrected N-value to determine the expected volumetric strain of saturated sands subjected to earthquake shaking. This analysis is also documented on the spreadsheets included in Appendix F.

Conclusions and Recommendations

The results of the liquefaction evaluation have identified liquefiable soils at all three of the boring locations. Liquefiable soils were encountered at Boring No. B-7 at depths between 18 and 31± feet, at Boring No. B-8 between depths of 43 and 48± feet, and boring No. B-9 at depths between 18 and 43± feet. Soils which are located above the historic groundwater table (18 feet), or possess factors of safety of at least 1.3, are considered to be non-liquefiable. Most of the silty clay and clayey silt strata encountered at the boring locations are considered non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the requirements of Special Publication 117A. Settlement analyses were conducted for each of the potentially liquefiable strata.

Based on the settlement analysis (also tabulated on the spreadsheets in Appendix F) total dynamic (liquefaction induced) settlements of 2.64± inches, 1.62± inches, and 1.95 inches could be expected at Boring Nos. B-7, B-8, and B-9, respectively. The associated differential settlement is considered to be to be up to one-half of the total settlement value, or 1.3 inches. The estimated differential settlement can be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of less than 0.002 inches per inch. This settlement is considered to be within the structural tolerances of typical buildings supported on conventional foundation systems. However, it should be noted that minor to moderate repairs, including repair of damaged drywall and stucco, etc., could be required after the occurrence of liquefaction-induced settlements.

Based on our understanding of the proposed development and the client's risk tolerances, it is considered feasible to support the proposed buildings on shallow foundation systems. Such foundation systems can be designed to resist the effects of the anticipated differential settlements, to the extent that the structures would not catastrophically fail. Designing the proposed buildings to remain completely undamaged during a major seismic event is not considered to be economically feasible. Based on this understanding, the use of shallow foundation systems is considered to be the most economical means of supporting the proposed residential buildings.

In order to support the proposed buildings on shallow foundations (such as spread footings) the structural engineer should verify that the structure would not catastrophically fail due to the predicted total and differential settlements. Any utility connections to the structures should be designed to withstand the estimated dynamic settlements. It should also be noted that minor to moderate repairs, including releveling, restoration of utility connections, repair of damaged drywall and stucco, etc., would likely be required after occurrence of the liquefaction-induced settlements.

The use of shallow foundation systems, as described in this report, is typical for buildings of these types, where they are underlain by the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the buildings at this site will also be typical of similar buildings in the vicinity of this project. However, if the owner determines that this level of potential damage is not acceptable, other geotechnical and structural options are available, including the use of ground improvement, deep foundations or a mat foundation.

6.2 Geotechnical Design Considerations

General

The subject site is generally underlain by loose to medium dense, variable strength native alluvial soils extending to depths of approximately 5 to 8± feet below the existing site grades. The results of consolidation/collapse testing performed for the referenced geotechnical investigation indicate that the upper portion (upper 5 to 6± feet) of the near surface alluvium possess unfavorable consolidation characteristics. Furthermore, some of the samples in the upper 5 to 8± feet were observed to be slightly to moderately porous. Therefore, remedial grading is recommended in order to remove a portion of the near surface alluvium, and replace these materials as a new, uniform layer of compacted structural fill beneath the foundations and floor slabs of the proposed structures.

As discussed in the previous section of this report, potentially liquefiable soils were identified at this site. The presence of the recommended layer of newly placed compacted structural fill above these liquefiable soils will help to reduce possible surface manifestations that could occur as a result of liquefaction. The foundation design recommendations presented in the subsequent sections of this report also contain recommendations to provide additional rigidity in order to reduce the potential effects of differential settlement that could occur as a result of liquefaction.

Settlement

The results of the consolidation/collapse testing indicate that the upper portion of the near surface native soils possess a moderate potential for collapse when exposed to moisture infiltration, and a moderate potential for consolidation when exposed to increases in the range of those that will be exerted by the foundations of the proposed structures. The recommended remedial grading will remove these soils and replace them as compacted structural fill. Following completion of the recommended grading, the post-construction settlements that could occur due to the near surface soils are expected to be within the structural tolerances of the proposed buildings.

Soluble Sulfates

The results of the soluble sulfate testing, as discussed in Section 5.0 of this report, indicate soluble sulfate concentrations of 0.120 and 0.130 percent. These concentrations are considered to be moderate with respect to the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Additionally, based on the results of previously performed laboratory testing for the referenced geotechnical report, soils with negligible to severe soluble sulfate concentrations were also encountered at the subject site. Therefore, specialized sulfate resistant concrete mix designs will be necessary at this site. It is recommended that concrete which will come into contact with the on-site soils be designed using the follow characteristics:

- Cement Type: V (Five)
- Minimum Compressive Strength (f'_c) = 4,500 lbs/in^2
- Maximum Water/Cement Ratio: 0.45

It is recommended that additional sulfate testing be performed at the completion of rough grading to verify the concentrations which are present in the actual building pad subgrade soils.

Expansion

The near surface soils at this site generally consist of silty sands and fine sandy silts. Results of laboratory testing indicates that these materials have are non-expansive (EI = 0). Therefore, no design considerations related to expansive soils are considered warranted for this project. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Shrinkage/Subsidence

Based on the results of the laboratory testing, removal and recompaction of the near-surface alluvium is estimated to result in an average shrinkage of 14 to 16 percent. Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1 feet.

These estimates are based on previous experience and the subsurface conditions encountered at the test boring locations. The actual amount of subsidence is expected to be variable and will be dependant on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

As discussed previously, detailed foundation plans and grading plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

Initial site stripping should include removal of any surficial vegetation and organic debris. These materials should be disposed of off-site. Based on the conditions at the time of our subsurface exploration, site stripping is expected to be limited to removal of walnut trees, other scattered trees, bushes, and areas of native grass and weed growth. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pads

Remedial grading is recommended within the new building pad areas to remove the near-surface, variable strength, collapsible alluvium and replace these materials as compacted structural fill. The upper portion of the native soils within the proposed building areas should be removed to a depth of at least 5 feet below the existing site grades. In order to provide a uniform subgrade for support of the proposed buildings, it is also recommended that the overexcavation extend to a depth of at least 5 feet below the proposed building pad subgrade elevations. Deeper removals of unsuitable soils may be necessary where loose, moderately porous, alluvial soils are encountered.

The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters and to an extent equal to the depth of fill below the foundation. If the proposed structures incorporate any exterior columns (such as for a canopy or overhang) the overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structures. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, or low density native soils are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, moisture treated to 2 to 4 percent above optimum moisture content, and compacted. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls should be overexcavated to a depth of 2 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pads.

The foundation areas for non-retaining site walls should be overexcavated to a depth of 2 feet below proposed foundation bearing grade. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Interior Streets

it is recommended that the upper portion of the native soils within the proposed street areas be removed to a depth of at least 2 feet below the existing site grades. In order to provide a uniform subgrade for support of the proposed improvements, it is also recommended that the overexcavation extend to a depth of at least 2 feet love the proposed subgrade elevations.

The subgrade soils should then be scarified to a depth of 12 ± inches, moisture conditioned to 2 to 4 percent above the optimum moisture content, and recompacted to at least 90 percent of the ASTM D–1557 maximum dry density

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent of the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.

- All grading and fill placement activities should be completed in accordance with the requirements of the CBC and the grading code of the city of Lake Elsinore.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of very low to low (EI < 50), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Lake Elsinore. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of fine sandy silts and silty fine sands. These materials are expected to be subject to minor caving within shallow excavations. Where caving occurs in shallow excavations, flattened excavation slopes (1.5h:1v) may be sufficient to provide excavation stability. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

Most of the near surface soils possess appreciable silt content and may become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. In addition, based on their granular content, some of the on-site soils will also be susceptible to erosion. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

Groundwater

Based on the conditions encountered in the borings, the groundwater table is considered to have existed at a depth of 37± feet in the southern portion of the site, and at a depth greater than 51½± feet in the northern portion of the site. Therefore it is not expected that groundwater will affect excavations for the foundations or utilities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pads will be underlain by structural fill soils used to replace the existing compressible native soils. The new structural fill soils are expected to extend to a depth of at least 5 feet below proposed building pad subgrade elevation, or approximately 3½ feet below nominal foundation bearing grade. Based on this subsurface profile, and based on the design considerations presented in Section 6.1 or this report, the proposed buildings may be supported on shallow foundation systems.

Building Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,000 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Four (4) No. 5 rebars (2 top and 2 bottom) due to the presence of potentially liquefiable soils.
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 24 inches below adjacent grade.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice, give the magnitude of predicted seismically induced settlements, and the structure type proposed for this site. Additional rigidity may be necessary for structural considerations, or to resist the effects of the seismically-induced settlements discussed in Section 6.1. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Within the new building areas, soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill or competent native alluvial soils, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 2 to 4 percent of the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively, under static conditions. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report. However, the likelihood of these two settlements combining is considered remote. The static settlements are expected to occur in a relatively short period of time after the building loads being applied to the foundations, during and immediately subsequent to construction. It should be noted that the projected potential dynamic settlement is related to a major seismic event and a conservative historic high groundwater level.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 2,500 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floors of the new structures may be constructed as conventional slabs-on-grade supported on newly placed structural fill, extending to a depth of at least 5 feet below finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 5 inches.
- Minimum slab reinforcement: Reinforcement of the floor slab should consist of No. 4 bars at 18-inches on center in both directions due to the presence of potentially liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.
- Slab underlayment should consist of a moisture vapor barrier constructed below the entire areas of proposed slabs-on-grade. In areas where the moisture sensitive floor coverings are not expected, such as garages, the moisture vapor barrier may be omitted. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview.
- Moisture condition the floor slab subgrade soils to 2 to 4 percent of the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.
- The floor slab should be structurally connected to the foundations as detailed by the structural engineer.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement. The steel reinforcement recommendations presented above are based on standard geotechnical practice, given the magnitude of predicted liquefaction settlement for this site. Additional rigidity may be necessary for structural considerations, or to resist the effects of the seismically-induced differential settlements discussed in Section 6.1 of this report.

6.7 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for patios, sidewalks and driveways should be prepared in accordance with the recommendations contained in the ***Grading Recommendations*** section of this report. Based on geotechnical considerations, exterior slabs on grade may be designed as follows:

- Minimum slab thickness: 4 inches
- Minimum slab reinforcement: Driveway slabs or other flatwork which may be subjected to vehicular traffic should include conventional welded wire mesh (6x6–W1.4xW1.4 WWF). Reinforcement in other exterior flatwork is not required, with respect to geotechnical conditions.
- Moisture condition the flatwork subgrade soils to a moisture content of 2 to 4 percent above the optimum moisture content, to a depth of at least 12 inches.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.
- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 4 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking and/or movement of exterior concrete slabs on grade should be expected.
- Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

Thickened Edges

Where the outer edges of concrete flatwork are to be bordered by landscaping, thickened edges should be used to prevent excessive infiltration and accumulation of water under the slabs. Thickened edges, if used, should be 6 to 8 inches wide, extend 12 inches below the tops of the finish slab surfaces, and be reinforced with a minimum of two No. 4 bars, one top and one bottom. Thickened edges are not mandatory; however, their inclusion in flatwork construction adjacent to landscaped areas will significantly reduce the potential for vertical and horizontal movement and subsequent cracking of the flatwork related to soil movement.

6.8 Landscape Wall Construction

Foundations

Foundations for landscape walls should be founded at a minimum depth of 12 inches below the lowest adjacent final grade. The footings should also be reinforced with a minimum of two No. 4 bars, one top and one bottom.

Construction Joints

In order to minimize the potential for unsightly cracking related to the effects of differential movements, construction joints should be provided in the walls at horizontal intervals of approximately 20± feet, and at each corner. The separations should be provided in the blocks and should not extend through the foundation. Foundations should be poured monolithically with continuous reinforcement along the entire length of the wall. A joint to provide positive separation between the wall face and adjacent flatwork is also recommended. A ½± inch thick felt joint may be used for this application.

6.9 Planters and Planter Walls

Area drains should be extended into all planters that are located within 5 feet of building walls, foundations, retaining walls and landscape walls to minimize infiltration of water into the adjacent foundation soils. The surface of the ground in these areas should also be sloped at a minimum gradient of 2 percent away from the walls and foundations. A drip irrigation system is also recommended to prevent overwatering and subsequent saturation of the foundation walls.

Planter walls should be supported by continuous concrete footings designed and constructed in accordance with the recommendations presented for landscape walls.

6.10 Retaining Wall Design and Construction

The site plan provided to our office does not indicate any retaining walls as part of the proposed development. However, in the event that retaining walls may be required, the following recommendations for retaining wall construction have been provided. It is assumed that in the event that retaining walls will be required, they will be limited in height to 5± feet. The parameters recommended for use in the design of retaining walls at the subject site are presented below:

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. The following parameters assume that only the on-site soils will be utilized for retaining wall backfill. The near surface soils generally consist of silty fine sands and fine sandy silts. Based on their composition, the on-site soils have been assigned a friction angle of 30 degrees.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: On-Site Silty Sands and Sandy Silts
Internal Friction Angle (ϕ)		30°
Unit Weight		125 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	42 lbs/ft^3
	Active Condition (2h:1v backfill)	67 lbs/ft^3
	At-Rest Condition (level backfill)	63 lbs/ft^3

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 2 feet below the proposed bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

It is recommended that a prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, be placed against the face of the retaining walls. The drainage composite should be installed in accordance with the manufacturer's specifications and extend from the top of the retaining wall footing to within 1 foot of the ground surface on the back side of the retaining wall. If the backfill soils are not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557-91). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Seismic Lateral Earth Pressures

In accordance with the 2013 CBC, any walls retaining 6 or more feet (in height) of soil must be designed for seismic lateral earth pressures. If walls retaining 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

6.11 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, extending to at least 2 feet below subgrade. The near surface soils generally silty fine sands, and fine sandy silts. These soils are considered to possess fair to good pavement support characteristics with estimated R-values of 30 to 40. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed R-value of 30. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

The recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base are presented below. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20 year design life, assuming six operational traffic days per week.

Traffic Index	No. of Heavy Trucks per Day
4.5	0
5.5	2

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

ASPHALT PAVEMENTS (R = 30)		
Materials	**Thickness (inches)**	
	Interior cul-de-sacs (TI = 4.5)	Interior Collector Streets (TI = 5.5)
Asphalt Concrete	3	3½
Aggregate Base	5	7
Compacted Subgrade (90% minimum compaction)	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS		
Materials	**Thickness (inches)**	
	Interior cul-de-sacs (TI = 4.5)	Interior Collector Streets (TI = 5.5)
PCC	5	5½
Compacted Subgrade (95% minimum compaction)	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. Reinforcing within all pavements should consist of at least heavy welded wire mesh (6x6-W2.9xW2.9 WWF) placed at mid height in the slab. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness. The actual joint spacing and reinforcing of the Portland cement concrete pavements should be determined by the structural engineer.

7.0 GENERAL COMMENTS

This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

8.0 REFERENCES

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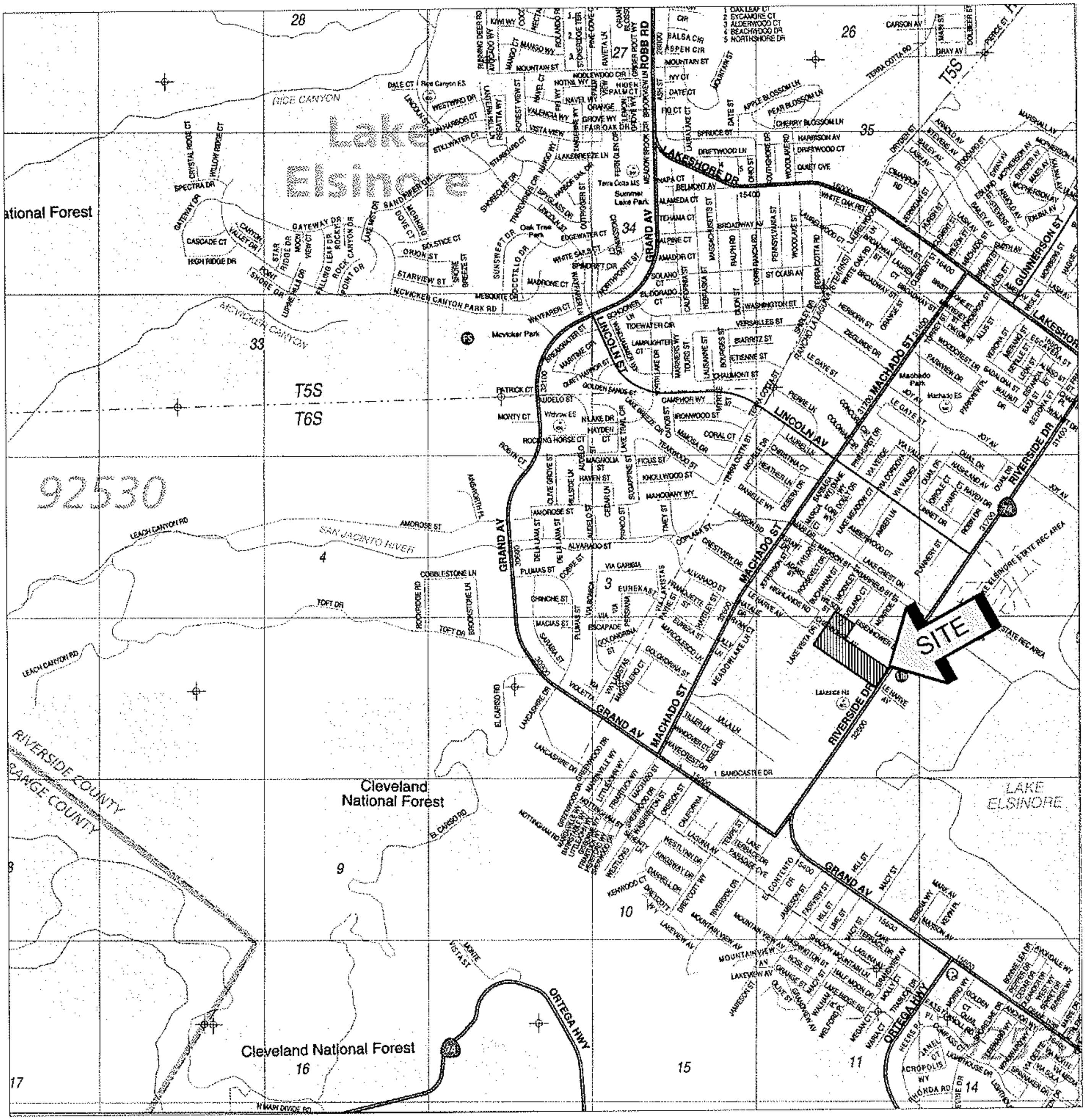
APPENDIX A



SOURCE: RIVERSIDE COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP

PROPOSED RESIDENTIAL DEVELOPMENT

LAKE ELSINORE, CALIFORNIA

SCALE: 1" = 2400'

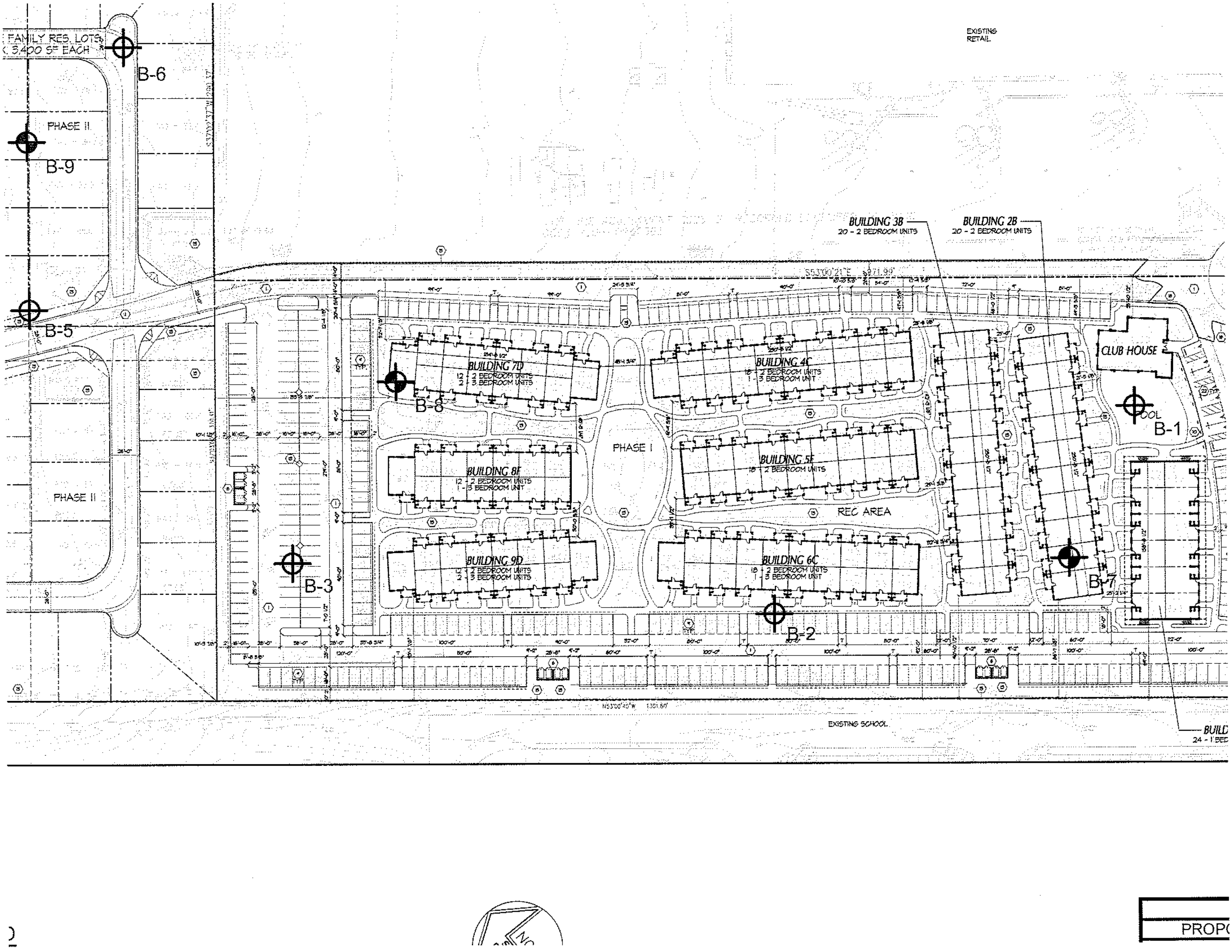
DRAWN: RF
CHKD: JAS

SCG PROJECT
14G178-1

PLATE 1

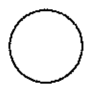
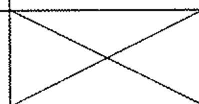
SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL


FAMILY RES. LOTS
X 3,400 SF EACH
B-6
PHASE II
B-9
B-5
PHASE II
B-3
EXISTING RETAIL
BUILDING 3B
20 - 2 BEDROOM UNITS
BUILDING 2B
20 - 2 BEDROOM UNITS
S53°00'21"E 971.99'
BUILDING 7D
12 - 2 BEDROOM UNITS
2 - 3 BEDROOM UNITS
B-8
BUILDING 4C
18 - 2 BEDROOM UNITS
1 - 3 BEDROOM UNIT
CLUB HOUSE
POOL
B-1
PHASE I
BUILDING 8F
12 - 2 BEDROOM UNITS
1 - 3 BEDROOM UNIT
BUILDING 5E
18 - 2 BEDROOM UNITS
REC AREA
BUILDING 9D
12 - 2 BEDROOM UNITS
2 - 3 BEDROOM UNITS
BUILDING 6C
18 - 2 BEDROOM UNITS
1 - 3 BEDROOM UNIT
B-7
B-2
N53°00'40"W 1301.60'
EXISTING SCHOOL
BUILD
24 - 1 BED
PROPO

APPENDIX B

BORING LOG LEGEND

SAMPLE TYPE	GRAPHICAL SYMBOL	SAMPLE DESCRIPTION
AUGER		SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED)
CORE		ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK.
GRAB		SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED)
CS		CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED)
NSR		NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL.
SPT		STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED)
SH		SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED)
VANE		VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED.

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

BLOW COUNT: Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

POCKET PEN.: Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

GRAPHIC LOG: Graphic Soil Symbol as depicted on the following page.

DRY DENSITY: Dry density of an undisturbed or relatively undisturbed sample in lbs/ft^3.

MOISTURE CONTENT: Moisture content of a soil sample, expressed as a percentage of the dry weight.

LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT: The moisture content above which a soil behaves as a plastic.

PASSING #200 SIEVE: The percentage of the sample finer than the #200 standard sieve.

UNCONFINED SHEAR: The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-7

JOB NO.: 14G178
PROJECT: Prop. Residential Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 8/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: 37 feet
CAVE DEPTH: 33 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Brown Silty fine Sand to fine Sandy Silt, slightly porous, loose to meidum dense-damp							
		16				89	8					
		14				90	8					
5					Gray Brown fine Sandy Silt, porous, loose-damp to moist							
		10				91	14					
					Gray Brown Silty fine Sand, medium dense-damp to moist							
		21				107	11					
					Dark Gray Brown Clayey Silt, porous, stiff-very moist							
10		14	2.25			87	28					
					Gray Brown Silty fine Sand to fine Sandy Silt, medium dense-moist to very moist							
15		16					15			48		
					Dark Gray Brown fine Sandy Silt, trace Iron oxide staining, medium dense-very moist							
20		13					18			78		
					Gray Brown fine Sandy Silt, medium dense-very moist							
25		14					18			61		
			4.0		Gray Brown Silty Clay, little fine Sand, stiff-very moist		21	33	15	72		
					Dark Gray fine Sandy Silt to Silty fine Sand, little Iron oxide staining, loose-very moist							
30		10	3.25				20			54		
			3.0		Dark Gray Brown Silty fine Sand to Silty Clay, little fine Sand, Iron oxide staining, stiff-very moist		24			73		
		21	3.0			107	22					
					Gray Brown Silty fine Sand to fine Sandy Silt, trace Iron oxide							

TBL 14G178.GPJ SOCALGEO.GDT 9/18/14

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-7

JOB NO.: 14G178
PROJECT: Prop. Residential Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 8/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: 37 feet
CAVE DEPTH: 33 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	29	1.5		staining, medium dense-very moist		19					
					Gray Brown Silty fine Sand to fine Sandy Silt, trace Iron oxide staining, medium dense-very moist							
					Gray fine to coarse Sand, some fine to coarse Gravel, dense-wet							
40	X	38					9			85		
					Dark Gray Clayey Silt, little fine Sand, stiff-wet							
45	X	14	2.75				29	35	16			
			4.0		Dark Gray fine Sandy Silt, little Clay, stiff-wet		23			59		
50	X	19	1.5				44	58	31	69		

Boring Terminated at 51½'

TBL 14G178.GPJ SOCALGEO.GDT 9/18/14

TEST BORING LOG **PLATE B-7b**

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-8

JOB NO.: 14G178
PROJECT: Prop. Residential Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 8/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 35 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Silty fine Sand, medium dense-dry to damp							
	X	11					6					
5	X	15			Gray Brown Silty fine Sand to fine Sandy Silt, medium dense-damp		8					
	X	9			Gray Brown Silty fine Sand, loose-damp to moist		9					
10	X	10	2.25		Gray Brown Silt, trace Clay, little fine Sand, stiff-moist		14					
15	X	10	1.5				20			81		
	X	22	\			106	17					
					Dark Gray Brown Silty Clay, little fine Sand, trace Iron oxide staining, stiff-very moist							
20	X	11	2.25				21	32	20	80		
					Dark Brown Silty fine to medium Sand, trace to little fine to coarse Gravel, medium dense-moist							
25	X	21					8			28		
					Gray Brown fine Sandy Silt, trace Iron oxide staining, trace fine Gravel, medium dense-very moist		18			65		
30	X	26					16			65		
					Gray Brown Silty fine to medium Sand, trace coarse Sand, little to some fine to coarse Gravel, dense-damp to moist							

TBL 14G178.GPJ SOCALGEO.GDT 9/18/14

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-8

JOB NO.: 14G178
PROJECT: Prop. Residential Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 8/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 35 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	43			Gray Brown Silty fine to medium Sand, trace coarse Sand, little to some fine to coarse Gravel, dense-damp		4					
					Gray Brown fine to coarse Sand, some fine to coarse Gravel, very dense-damp							
40	X	79					3					
					Dark Gray Brown Silty Clay, trace fine Sand, stiff-very moist							
45	X	14	2.0				26	36	19			
					Gray Brown Silty fine Sand, medium dense-very moist		23			44		
					Dark Gray Clayey Silt, little fine Sand, hard-very moist							
50	X	32	3.0				23			71		

Boring Terminated at 51½'

TBL 14G178.GPJ SOCALGEO.GDT 9/18/14

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-9

JOB NO.: 14G178
PROJECT: Prop. Residential Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 8/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 38 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Light Gray Brown Silty fine Sand, porous, loose-damp							
		9				77	5					
					Gray Brown Silty fine Sand, calcareous nodules and veins, slightly porous, medium dense-damp							
		40				97	7					
5												
		14				97	11					
					Gray Brown fine Sandy Silt, porous, loose-damp to moist							
		18				98	13					
10		15				96	13					
15		11					16			74		
					Gray Brown fine Sandy Silt, trace Iron oxide staining, very stiff-moist							
20		17					13			75		
					Gray Brown fine Sandy Clay, trace to little Silt, stiff-very moist							
25		13	2.25				20	29	18	81		
30		22	3.0				17			75		
					Gray Brown Silty fine Sand, trace fine to coarse Gravel, medium dense-damp to moist							

TBL 14G178.GPJ SOCALGEO.GDT 9/18/14

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-9

JOB NO.: 14G178
PROJECT: Prop. Residential Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 8/8/14
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 38 feet
READING TAKEN: At Completion

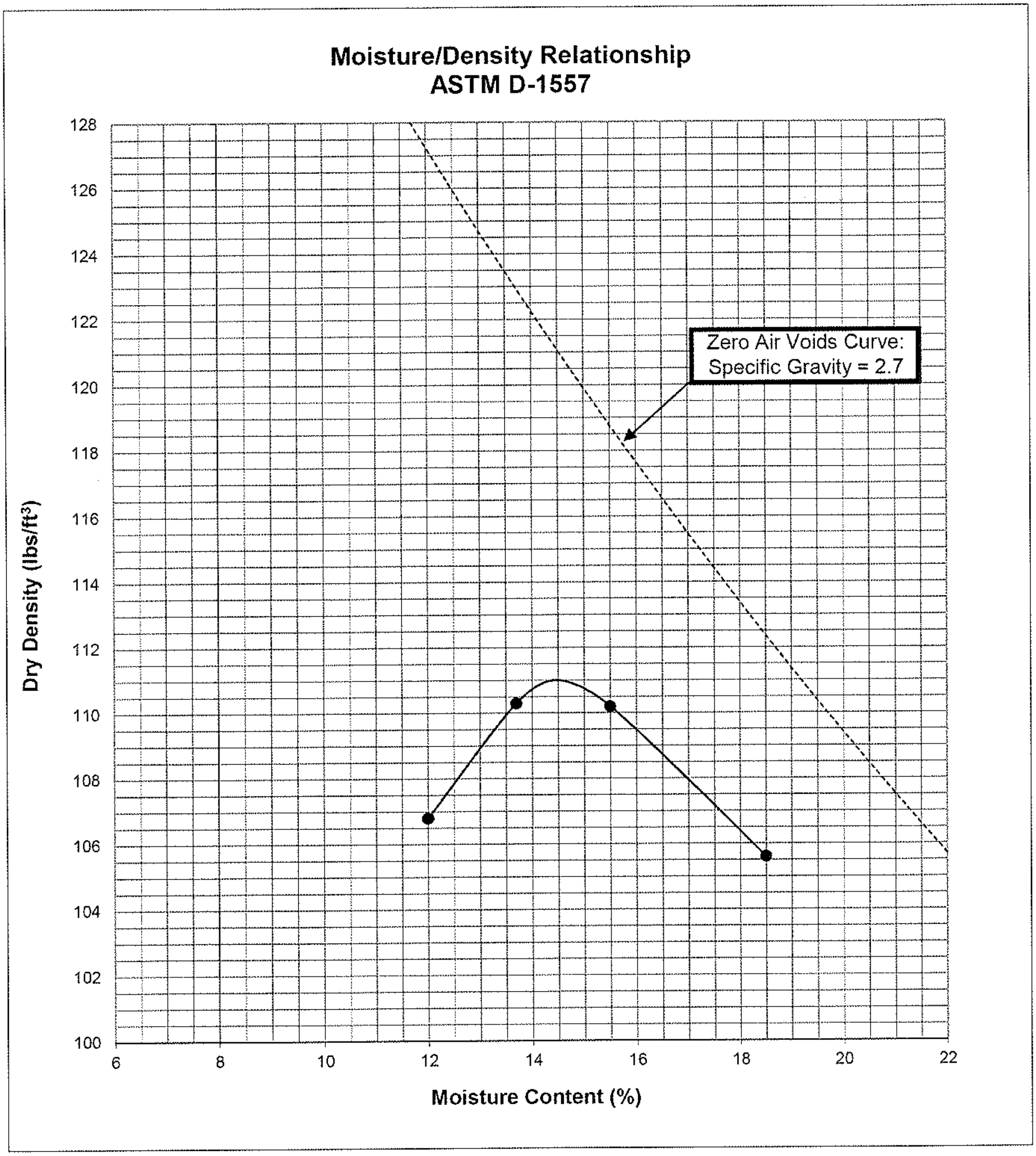
FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	25					7			33		
			4.0		Gray Brown fine Sandy Clay, trace to little Silt, trace iron oxide staining, very stiff-moist to very moist		18	30	19	70		
					Dark Gray Brown fine Sandy Silt, little Clay, trace Iron oxide staining, very stiff-very moist							
40	X	22					19			71		
					Dark Gray Silty fine Sand, trace medium Sand, trace fine Gravel, dense-very moist							
45	X	31					14					
					Dark Gray fine Sandy Silt, hard-very moist							
50	X	42					21					
					Dark Gray fine to coarse Sand, some fine to coarse Gravel, trace Silt, dense-moist		6					

Boring Terminated at 51½'

TBL 14G178.GPJ SOCALGEO.GDT 9/18/14

TEST BORING LOG

APPENDIX C



Soil ID Number		B-7 @ 0 to 5'
Optimum Moisture (%)		14.5
Maximum Dry Density (pcf)		111
Soil Classification	Gray Brown fine Sandy Silt, little Clay	

Proposed Residential Development
Lake Elsinore, California
Project No. 14G178
PLATE C-1

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX D

GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.

- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.

- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.

- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.

- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.

- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.

- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

<u>Compacted Fills</u>

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:

  - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.

  - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

- Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.

- To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates D-6.

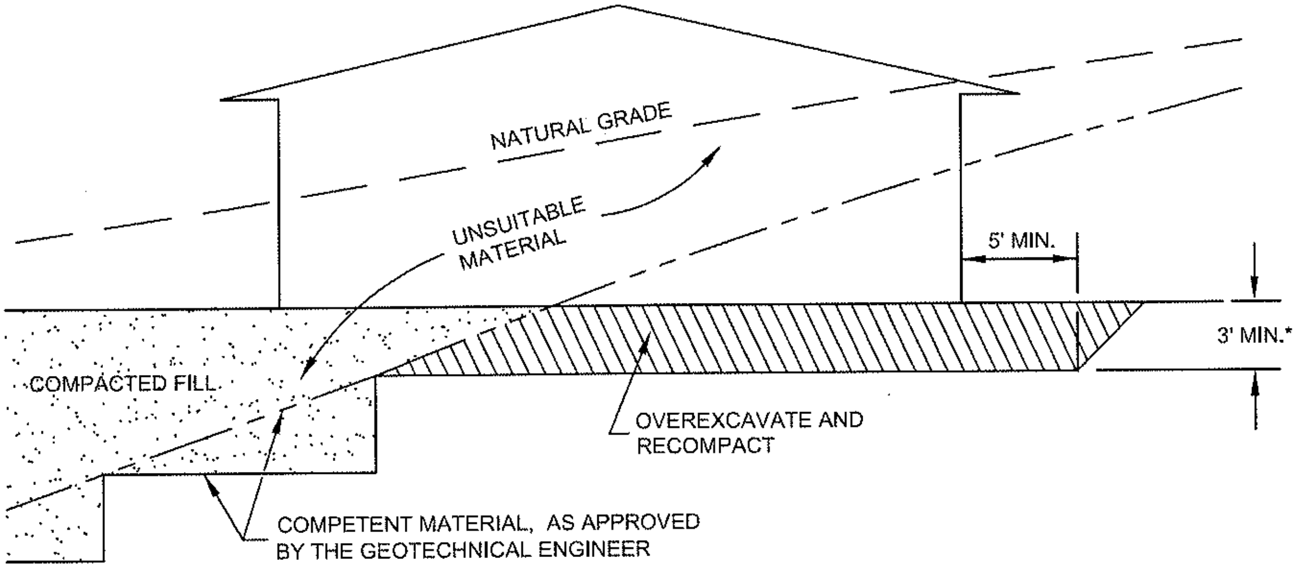
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

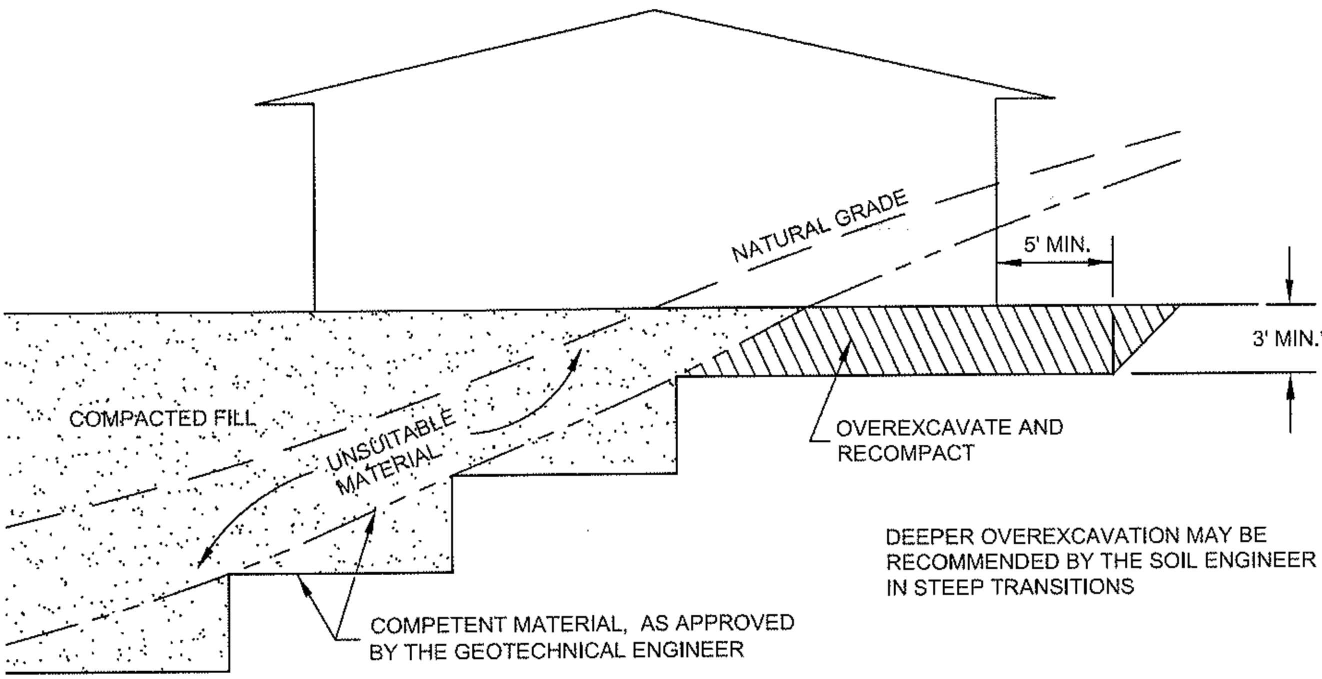
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

CUT LOT



CUT/FILL LOT (TRANSITION)



*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION. ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

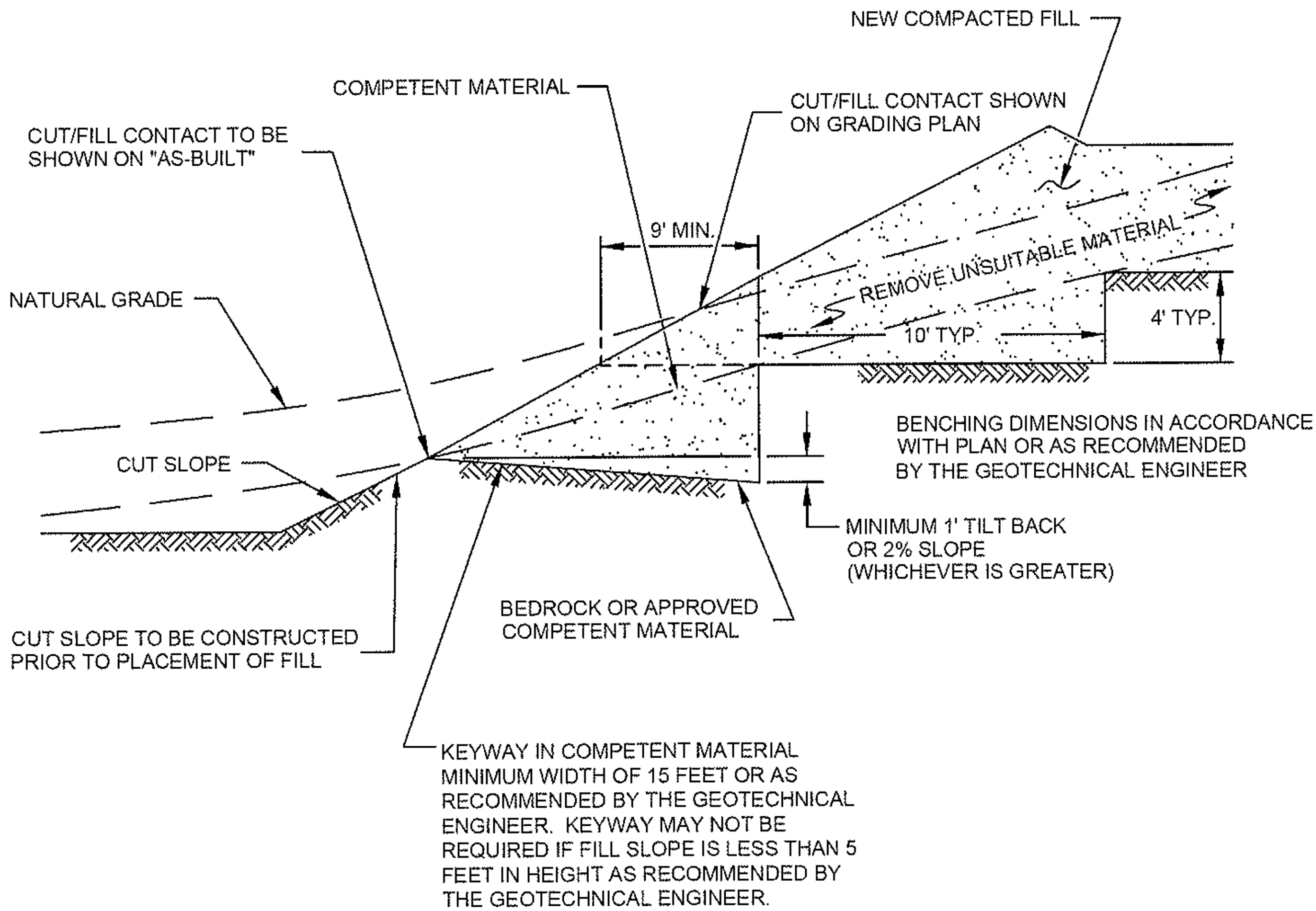
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

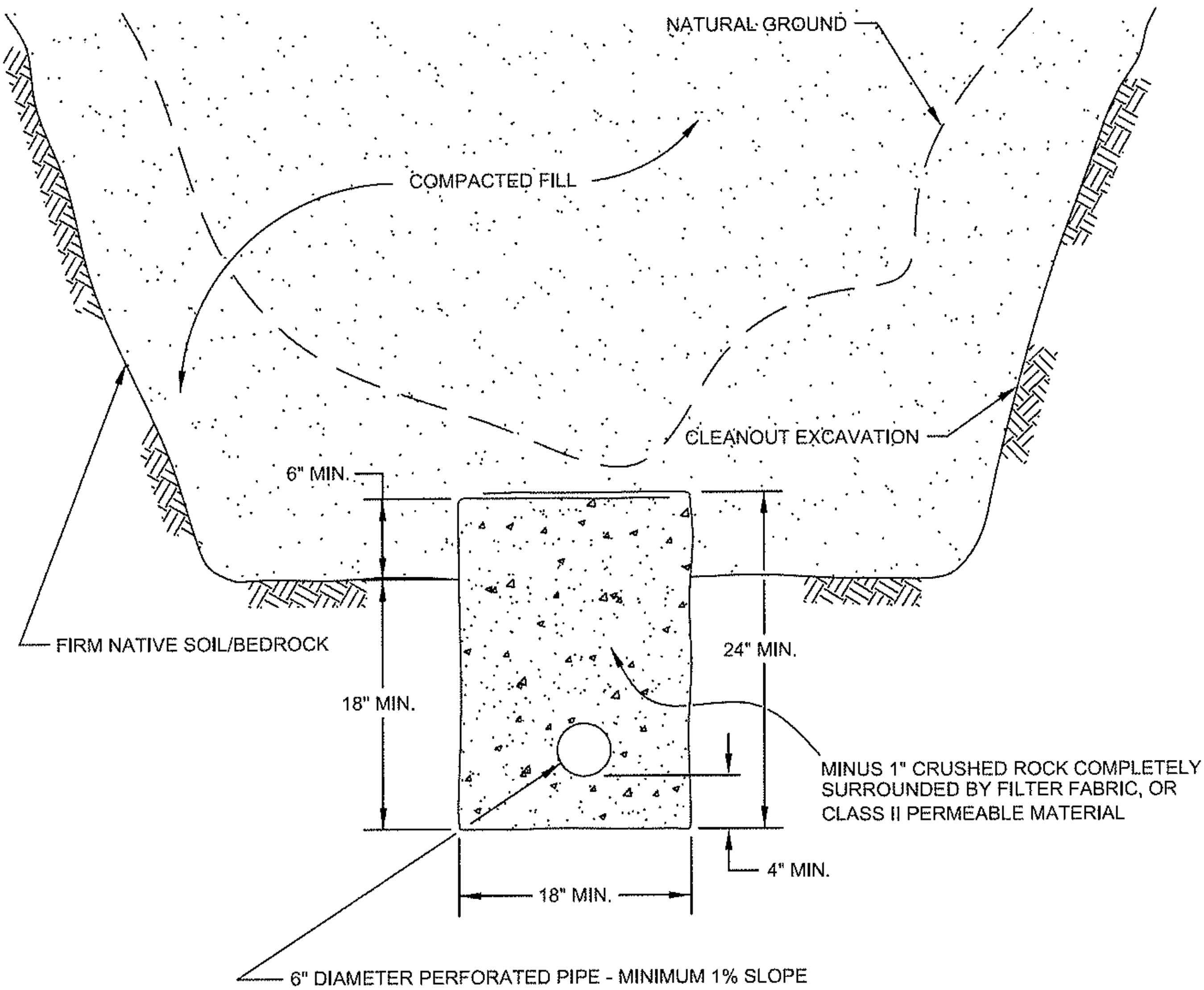
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



PIPE MATERIAL	DEPTH OF FILL OVER SUBDRAIN
ADS (CORRUGATED POLETHYLENE)	8
TRANSITE UNDERDRAIN	20
PVC OR ABS: SDR 35	35
SDR 21	100

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

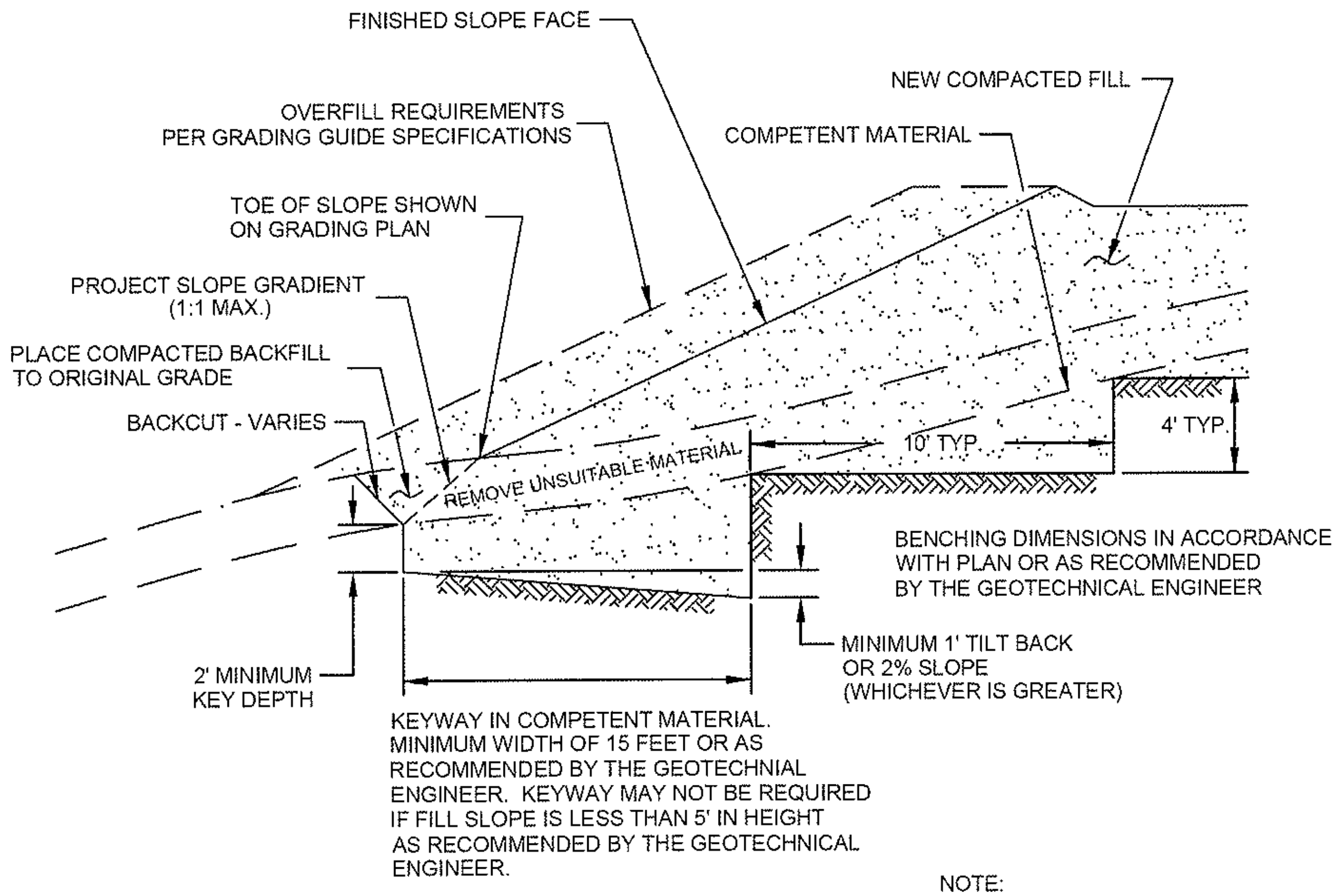
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SOUTHERN CALIFORNIA GEOTECHNICAL



NOTE:
BENCHING SHALL BE REQUIRED
WHEN NATURAL SLOPES ARE
EQUAL TO OR STEEPER THAN 5:1
OR WHEN RECOMMENDED BY
THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

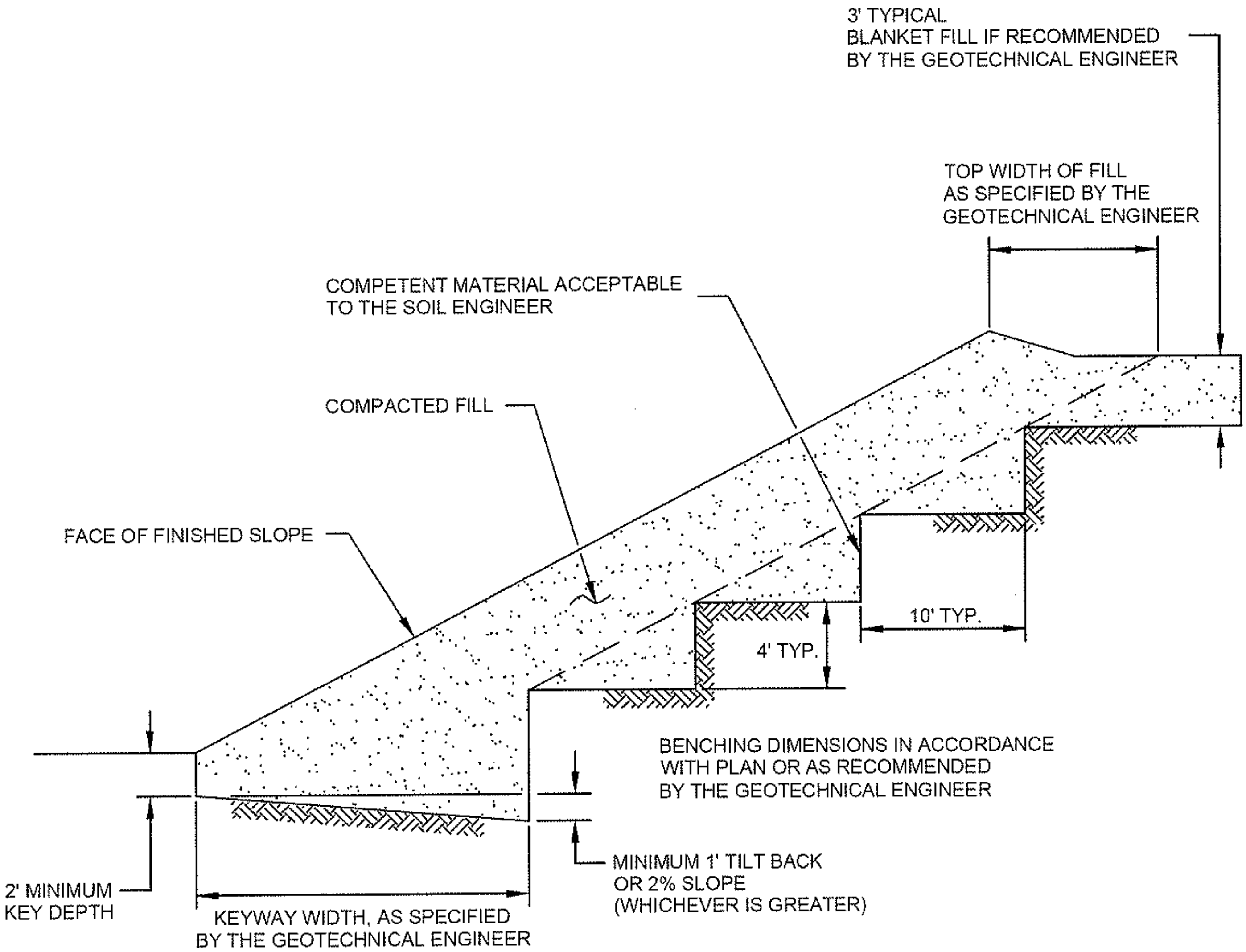
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

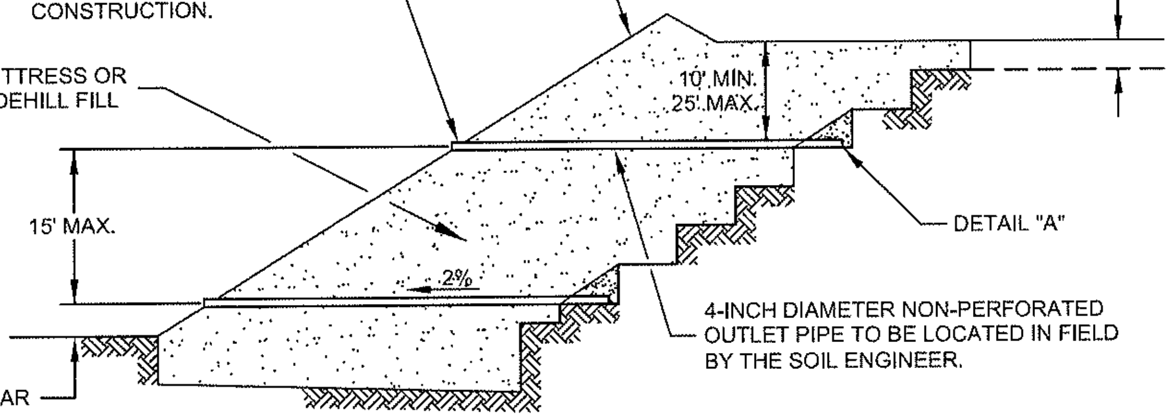
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL



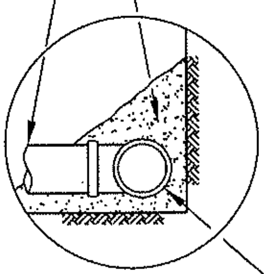
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

NOTES:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

SLOPE FILL SUBDRAINS

GRADING GUIDE SPECIFICATIONS

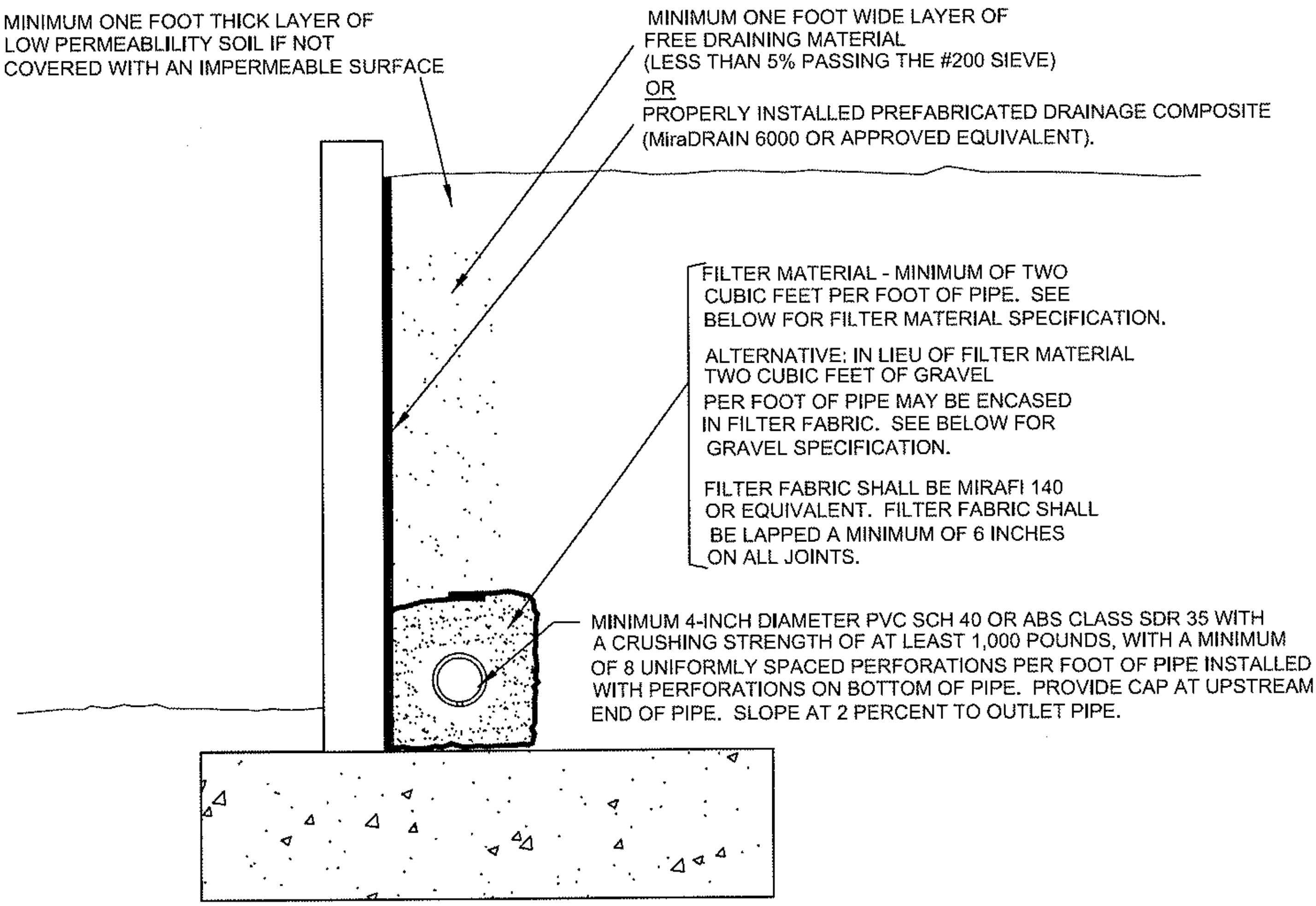
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

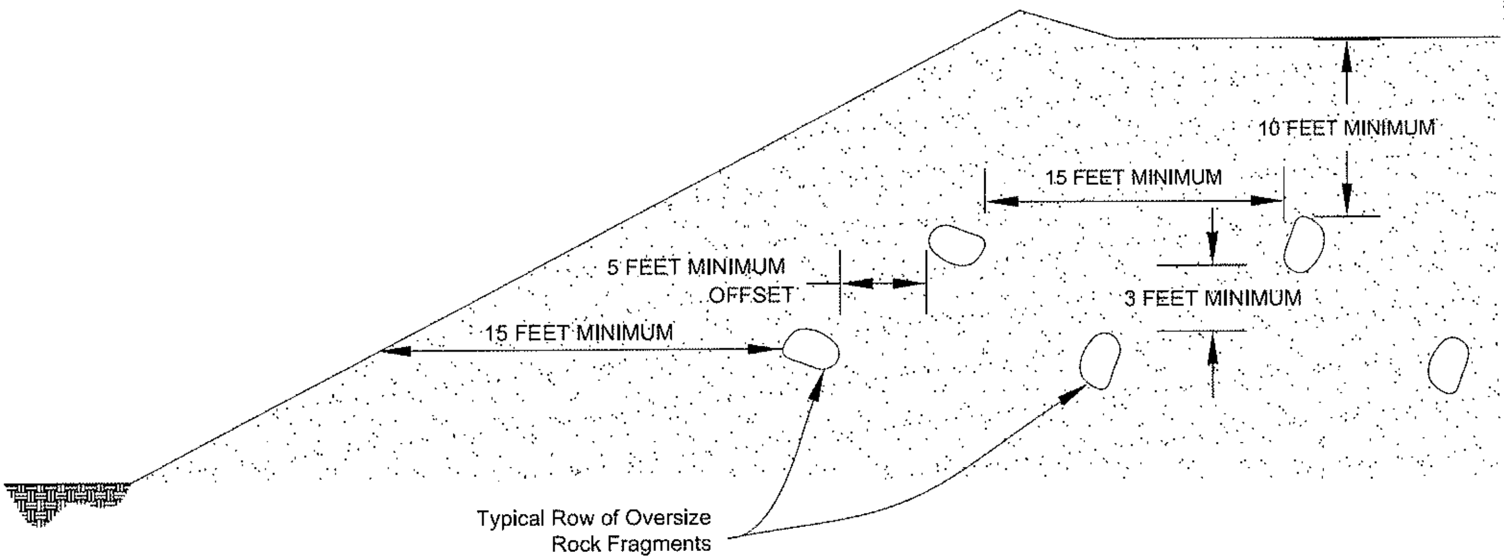
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

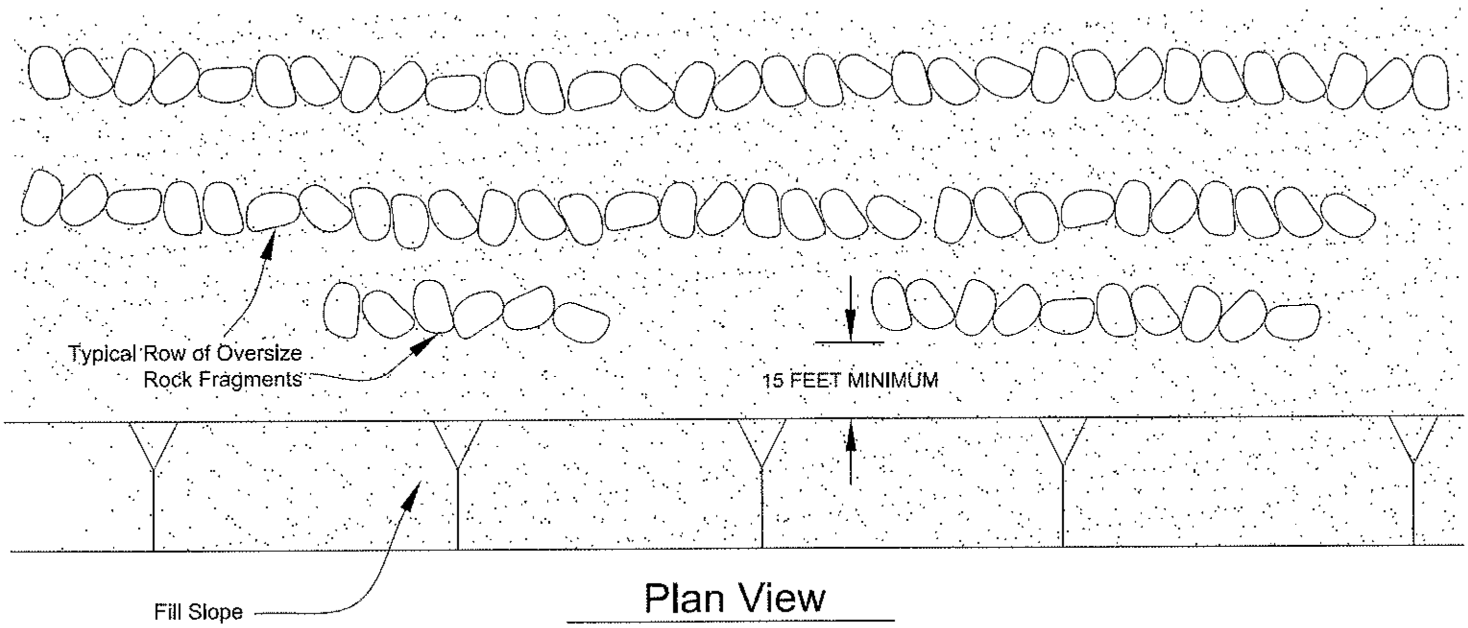
DRAWN: JAS
CHKD: GKM

PLATE D-7

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

USGS Design Maps Summary Report

User–Specified Input

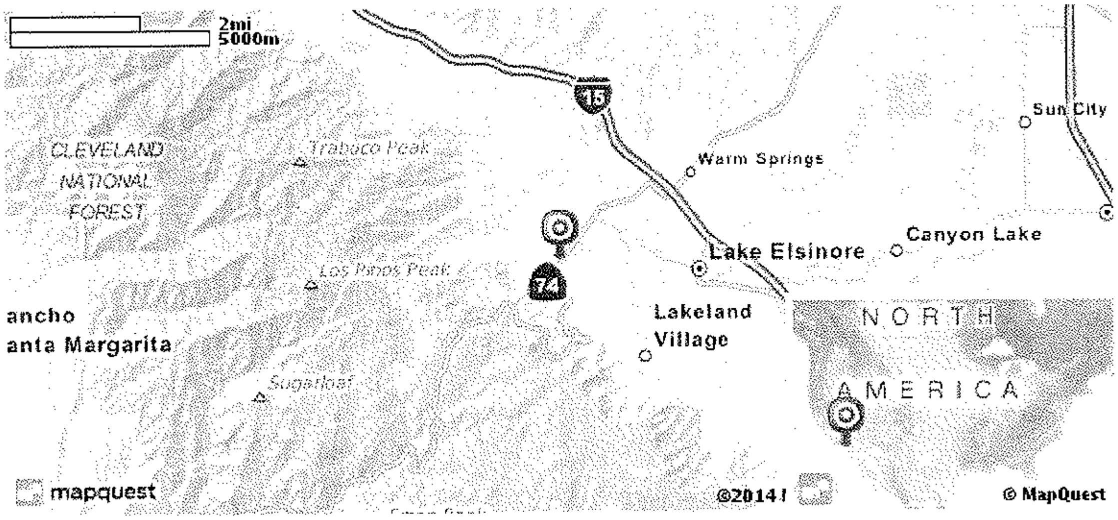
Report Title Proposed Residential Development
Wed August 13, 2014 22:50:59 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 33.67087°N, 117.37956°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category I/II/III



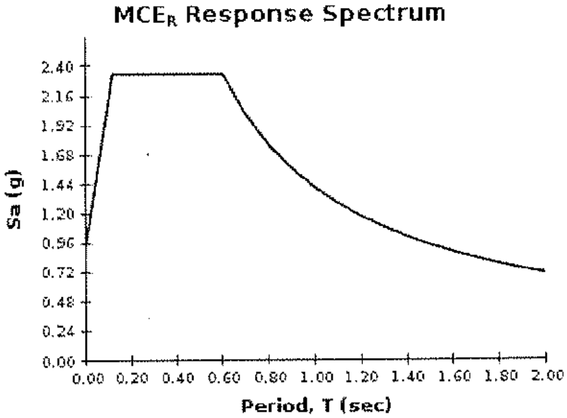
USGS–Provided Output

$S_S =$ 2.333 g $S_{MS} =$ 2.333 g $S_{DS} =$ 1.556 g

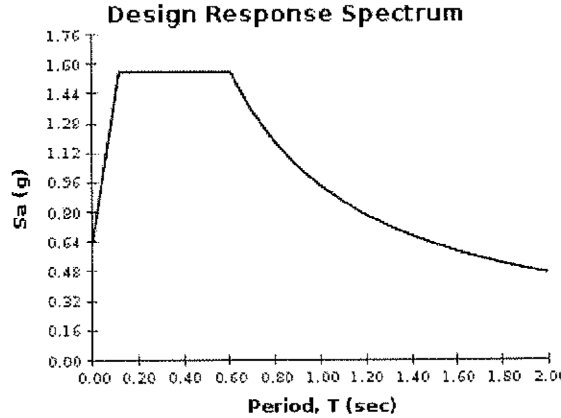
$S_1 =$ 0.940 g $S_{M1} =$ 1.410 g $S_{D1} =$ 0.940 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

SEISMIC DESIGN PARAMETERS
PROPOSED RESIDENTIAL DEVELOPMENT
LAKE ELSINORE, CALIFORNIA

DRAWN: RF
CHKD: JAS
SCG PROJECT
14G178-1
PLATE E-1

SoCalGeo
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CALIFORNIA
GEOTECHNICAL

Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.931

Equation (11.8-1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.931 = 0.931$ g

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.931 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] $C_{RS} = 0.909$

From Figure 22-18 [6] $C_{R1} = 0.897$

SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

MCE PEAK GROUND ACCELERATION
PROPOSED MFR DEVELOPMENT
LAKE ELSINORE, CALIFORNIA

DRAWN: RF
CHKD: JAS
SCG PROJECT
14G178-1
PLATE E-2

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION 2014

Project Name	Residential Development
Project Location	Lake Elsinore, CA
Project Number	14G178
Engineer	DWN
Boring No.	B-7

MCE_G Design Acceleration	0.931	(g)
Design Magnitude	6.96	
Historic High Depth to Groundwater	18	(ft)
Current Depth to Groundwater	37	(ft)
Borehole Diameter	8	(in)
Calculated Magnitude Scaling Factor (8)	1.15	

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.96)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(9)	(10)	(11)	(12)	(13)		
5.5	0	13	6.5		120		1.3	1.15	1	1.60	0.75	0.0	0.0	780	780	780	0.99	1.05	N/A	N/A	0.60	N/A	Above Water Table
16	13	18	15.5	16	120	48	1.3	1.15	1.21	1.04	0.85	25.5	31.1	1860	1860	1860	0.95	1.03	0.57	0.67	0.57	N/A	Above Water Table
21	18	23	20.5	13	120	78	1.3	1.15	1.17	0.90	0.95	19.4	25.0	2460	2304	2460	0.93	0.98	0.29	0.33	0.60	0.55	**Liquefiable**
25.5	23	25.5	24.25	14	120	61	1.3	1.15	1.16	0.83	0.95	19.2	24.8	2910	2520	2910	0.91	0.97	0.29	0.32	0.63	0.50	**Liquefiable**
26	25.5	28	26.75	14	120	72	1.3	1.15	1.16	0.79	0.95	18.2	23.7	3210	2664	3210	0.89	0.96	0.26	0.29	0.65	N/A	NonLiq: PI > 18
30.5	28	31	29.5	10	120	54	1.3	1.15	1.11	0.75	0.95	11.8	17.4	3540	2822	3540	0.88	0.96	0.18	0.20	0.67	0.30	**Liquefiable**
31	31	34	32.5	10	120	73	1.3	1.15	1.1	0.72	0.95	11.2	16.8	3900	2995	3900	0.86	0.96	0.17	0.19	0.68	N/A	NonLiq: PI > 18
36	34	38	36	29	120		1.3	1.15	1.29	0.68	1	38.2	38.2	4320	3197	4320	0.84	0.88	2.00	2.00	0.69	2.90	Non-Liquefiable
41	38	43	40.5	38	120	8	1.3	1.15	1.3	0.66	1	48.5	48.8	4860	3456	4642	0.82	0.85	2.00	1.97	0.70	2.83	Non-Liquefiable
46	43	46	44.5	14	120	85	1.3	1.15	1.13	0.64	1	15.2	20.7	5340	3686	4872	0.79	0.92	0.22	0.23	0.70	N/A	NonLiq: PI > 18
46	46	48	47	14	120	59	1.3	1.15	1.13	0.63	1	15.0	20.6	5640	3830	5016	0.78	0.92	0.21	0.23	0.70	N/A	NonLiq: PI > 18
49	48	50	49	19	120	69	1.3	1.15	1.18	0.62	1	20.9	26.5	5880	3946	5131	0.77	0.89	0.33	0.34	0.69	N/A	NonLiq: PI > 18

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Lao and Whitman, 1986, $C_N = (2.0\ ksf / p'_o)^{1/2}$

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Magnitude Scaling Factor calculated by Eq. 51 (Boulanger and Idriss, 2008)

(9) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Residential Development
Project Location	Lake Elsinore, CA
Project Number	14G178
Engineer	DWN

Boring No. B-7

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
5.5	0	13	6.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	13.00		0.000		0.00	Above Water Table
16	13	18	15.5	25.5	5.6	31.1	N/A	0.04	-0.17	0.00	5.00		0.000		0.00	Above Water Table
21	18	23	20.5	19.4	5.6	25.0	0.55	0.09	0.23	0.09	5.00		0.019		1.14	**Liquefiable**
25.5	23	25.5	24.25	19.2	5.6	24.8	0.50	0.09	0.24	0.09	2.50		0.019		0.57	**Liquefiable**
26	25.5	28	26.75	18.2	5.6	23.7	N/A	0.10	0.31	0.00	2.50		0.000		0.00	NonLiq: PI > 18
30.5	28	31	29.5	11.8	5.6	17.4	0.30	0.21	0.65	0.21	3.00		0.026		0.93	**Liquefiable**
31	31	34	32.5	11.2	5.6	16.8	N/A	0.23	0.68	0.00	3.00		0.000		0.00	NonLiq: PI > 18
36	34	38	36	38.2	0.0	38.2	2.90	0.01	-0.67	0.00	4.00		0.000		0.00	Non-Liquefiable
41	38	43	40.5	48.5	0.4	48.8	2.83	0.00	-1.50	0.00	5.00		0.000		0.00	Non-Liquefiable
46	43	46	44.5	15.2	5.5	20.7	N/A	0.15	0.48	0.00	3.00		0.000		0.00	NonLiq: PI > 18
46	46	48	47	15.0	5.6	20.6	N/A	0.15	0.49	0.00	2.00		0.000		0.00	NonLiq: PI > 18
49	48	50	49	20.9	5.6	26.5	N/A	0.07	0.14	0.00	2.00		0.000		0.00	NonLiq: PI > 18
											Total Deformation (in)				2.64	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Voumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION 2014

Project Name	Residential Development
Project Location	Lake Elsinore, CA
Project Number	14G178
Engineer	DWN
Boring No.	B-8

MCE_G Design Acceleration	0.931	(g)
Design Magnitude	6.96	
Historic High Depth to Groundwater	18	(ft)
Current Depth to Groundwater	60	(ft)
Borehole Diameter	8	(in)
Calculated Magnitude Scaling Factor (8)	1.15	

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.96)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(9)	(10)	(11)	(12)	(13)		
5.5	0	13	6.5		120		1.3	1.15	1	1.60	0.75	0.0	0.0	780	780	780	0.99	1.05	N/A	N/A	0.60	N/A	Above Water Table
16	13	18	15.5	10	120	81	1.3	1.15	1.13	1.04	0.85	14.9	20.5	1860	1860	1860	0.95	1.02	0.21	0.25	0.57	N/A	Above Water Table
21	18	23	20.5	11	120	80	1.3	1.15	1.14	0.90	0.95	16.1	21.6	2460	2304	2460	0.93	0.99	N/A	N/A	N	N/A	Non-Liq:PI=12,w<.85LL
25.5	23	25.5	24.25	21	120	28	1.3	1.15	1.25	0.83	0.95	30.8	36.1	2910	2520	2910	0.91	0.95	1.41	1.55	0.63	2.44	Non-Liquefiable
26	25.5	28	26.75	21	120	65	1.3	1.15	1.24	0.79	0.95	29.1	34.7	3210	2664	3210	0.89	0.94	1.04	1.12	0.65	1.72	Non-Liquefiable
31	28	33	30.5	26	120	65	1.3	1.15	1.27	0.74	0.95	34.7	40.3	3660	2880	3660	0.87	0.91	2.00	2.00	0.67	2.98	Non-Liquefiable
36	33	38	35.5	43	120		1.3	1.15	1.3	0.69	1	57.3	57.3	4260	3168	4260	0.85	0.88	2.00	2.00	0.69	2.91	Non-Liquefiable
41	38	43	40.5	79	120		1.3	1.15	1.3	0.64	1	98.5	98.5	4860	3456	4860	0.82	0.85	2.00	1.97	0.70	2.83	Non-Liquefiable
46	43	46	44.5	14	120		1.3	1.15	1.13	0.61	1	14.4	14.4	5340	3686	5340	0.79	0.94	0.15	0.16	0.70	0.24	**Liquefiable**
46	46	48	47	14	120	44	1.3	1.15	1.12	0.60	1	14.0	19.6	5640	3830	5640	0.78	0.92	0.20	0.21	0.70	0.31	**Liquefiable**
51	48	50	49	32	120	59	1.3	1.15	1.28	0.58	1	35.7	41.3	5880	3946	5880	0.77	0.81	2.00	1.88	0.69	2.71	Non-Liquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Lao and Whitman, 1986, $C_N = (2.0\ \text{ksf} / p'_o)^{1/2}$

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Magnitude Scaling Factor calculated by Eq. 51 (Boulanger and Idriss, 2008)

(9) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Residential Development
Project Location	Lake Elsinore, CA
Project Number	14G178
Engineer	DWN

Boring No. B-8

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
5.5	0	13	6.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	13.00		0.000		0.00	Above Water Table
16	13	18	15.5	14.9	5.5	20.5	N/A	0.15	0.49	0.00	5.00		0.000		0.00	Above Water Table
21	18	23	20.5	16.1	5.5	21.6	N/A	0.13	0.43	0.00	5.00		0.000		0.00	Non-Liq:PI=12,w<.85LL
25.5	23	25.5	24.25	30.8	5.3	36.1	2.44	0.02	-0.52	0.00	2.50		0.000		0.00	Non-Liquefiable
26	25.5	28	26.75	29.1	5.6	34.7	1.72	0.02	-0.41	0.01	2.50		0.000		0.00	Non-Liquefiable
31	28	33	30.5	34.7	5.6	40.3	2.98	0.01	-0.83	0.00	5.00		0.000		0.00	Non-Liquefiable
36	33	38	35.5	57.3	0.0	57.3	2.91	0.00	-2.19	0.00	5.00		0.000		0.00	Non-Liquefiable
41	38	43	40.5	98.5	0.0	98.5	2.83	0.00	-5.92	0.00	5.00		0.000		0.00	Non-Liquefiable
46	43	46	44.5	14.4	0.0	14.4	0.24	0.29	0.78	0.29	3.00		0.030		1.06	**Liquefiable**
46	46	48	47	14.0	5.6	19.6	0.31	0.17	0.54	0.17	2.00		0.023		0.56	**Liquefiable**
51	48	50	49	35.7	5.6	41.3	2.71	0.01	-0.90	0.00	2.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				1.62	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Voumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION 2014

Project Name	Residential Development
Project Location	Lake Elsinore, CA
Project Number	14G178
Engineer	DWN
Boring No.	B-9

MCE_G Design Acceleration	0.931	(g)
Design Magnitude	6.96	
Historic High Depth to Groundwater	18	(ft)
Current Depth to Groundwater	60	(ft)
Borehole Diameter	8	(in)
Calculated Magnitude Scaling Factor (8)	1.15	

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.96)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(9)	(10)	(11)	(12)	(13)		
5.5	0	13	6.5		120		1.3	1.15	1	1.60	0.75	0.0	0.0	780	780	780	0.99	1.05	N/A	N/A	0.60	N/A	Above Water Table
16	13	18	15.5	11	120	84	1.3	1.15	1.14	1.04	0.85	16.6	22.1	1860	1860	1860	0.95	1.02	0.24	0.28	0.57	N/A	Above Water Table
21	18	23	20.5	17	120	75	1.3	1.15	1.22	0.90	0.95	26.5	32.1	2460	2304	2460	0.93	0.98	0.65	0.74	0.60	1.23	**Liquefiable**
26	23	28	25.5	13	120	81	1.3	1.15	1.15	0.81	0.95	17.2	22.7	3060	2592	3060	0.90	0.97	0.24	0.27	0.64	0.42	**Liquefiable**
31	28	33	30.5	22	120	75	1.3	1.15	1.23	0.74	0.95	28.4	34.0	3660	2880	3660	0.87	0.92	0.91	0.96	0.67	1.44	Non-Liquefiable
35.5	33	35.5	34.25	22	120	33	1.3	1.15	1.23	0.70	1	28.2	33.7	4110	3096	4110	0.85	0.91	0.85	0.89	0.68	1.30	Non-Liquefiable
36	35.5	38	36.75	25	120	70	1.3	1.15	1.25	0.67	1	31.5	37.1	4410	3240	4410	0.84	0.87	1.78	1.79	0.69	2.60	Non-Liquefiable
41	38	43	40.5	22	120	71	1.3	1.15	1.21	0.64	1	25.6	31.1	4860	3456	4860	0.82	0.89	0.56	0.58	0.70	0.84	**Liquefiable**
46	43	48	45.5	31	120		1.3	1.15	1.28	0.61	1	35.9	35.9	5460	3744	5460	0.79	0.84	1.35	1.31	0.70	1.88	Non-Liquefiable
51	48	50	49	42	120		1.3	1.15	1.3	0.58	1	47.6	47.6	5880	3946	5880	0.77	0.81	2.00	1.88	0.69	2.71	Non-Liquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Lao and Whitman, 1986, $C_N = (2.0 \text{ ksf} / p'_o)^{1/2}$

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Magnitude Scaling Factor calculated by Eq. 51 (Boulanger and Idriss, 2008)

(9) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Residential Development
Project Location	Lake Elsinore, CA
Project Number	14G178
Engineer	DWN

Boring No. B-9

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
5.5	0	13	6.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	13.00		0.000		0.00	Above Water Table
16	13	18	15.5	16.6	5.5	22.1	N/A	0.12	0.40	0.00	5.00		0.000		0.00	Above Water Table
21	18	23	20.5	26.5	5.6	32.1	1.23	0.03	-0.23	0.02	5.00		0.004		0.25	**Liquefiable**
26	23	28	25.5	17.2	5.5	22.7	0.42	0.12	0.37	0.12	5.00		0.021		1.24	**Liquefiable**
31	28	33	30.5	28.4	5.6	34.0	1.44	0.03	-0.36	0.01	5.00		0.000		0.00	Non-Liquefiable
35.5	33	35.5	34.25	28.2	5.5	33.7	1.30	0.03	-0.34	0.02	2.50		0.000		0.00	**Non-Liquefiable**
36	35.5	38	36.75	31.5	5.6	37.1	2.60	0.02	-0.59	0.00	2.50		0.000		0.00	Non-Liquefiable
41	38	43	40.5	25.6	5.6	31.1	0.84	0.04	-0.16	0.04	5.00		0.008		0.46	**Liquefiable**
46	43	48	45.5	35.9	0.0	35.9	1.88	0.02	-0.50	0.00	5.00		0.000		0.00	Non-Liquefiable
51	48	50	49	47.6	0.0	47.6	2.71	0.00	-1.40	0.00	2.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				1.95	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Voumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

APPENDIX G

Southern California Geotechnical

BORING NO. B-1

JOB NO.: 05G289
PROJECT: Proposed MFR Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 12/5/05
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joaquin Baca

WATER DEPTH: Dry
CAVE DEPTH: 20 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS							
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					1 to 2± inches Topsoil/Root mat material							
		12			ALLUVIUM: Dark Brown to Black Silty fine Sand to fine Sandy Silt, slightly porous, loose-damp to moist	91	25					
		11			Brown to Dark Brown fine Sandy Silt, loose-damp to moist	83	26					
5		8				90	16					
		15			Dark Brown Silty fine to medium Sand, loose to medium dense-damp to moist	102	14					
		21			Brown fine to medium Sand, trace Silt and fine Gravel, medium dense-damp	109	5					
10												
					Dark Brown fine Sandy Silt, medium dense-moist							
		12					18					
15												
					Dark Gray Brown Clayey fine Sand, loose-moist							
		10					22					
20												
		16					23					
25												
					Boring Terminated at 25'							

TBL 05G289.GPJ SOCALGEO.GDT 12/12/05

TEST BORING LOG

Southern California Geotechnical INC.

BORING NO. B-2

JOB NO.: 05G289	DRILLING DATE: 12/5/05	WATER DEPTH: 41 feet
PROJECT: Proposed MFR Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 29 feet
LOCATION: Lake Elsinore, California	LOGGED BY: Joaquin Baca	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					1 to 2± inches Topsoil/Root mat material							
	X	6			ALLUVIUM: Dark Brown to Black Silty fine Sand to fine Sandy Silt, trace fine root fibers, slightly porous, loose-damp to moist		16			68		
	X	8			Dark Brown fine Sandy Silt, trace calcareous veining, slightly porous, loose-moist		19			76		
5	X	5					22			77		
	X	9					20			68		
10												
					Dark Gray Brown fine Sandy Silt to Silty fine Sand, trace Clay, loose to medium dense-moist							
	X	10					19			64		
15												
	X	14					19			44		
20												
					Gray Brown fine Sandy Silt to Silty fine to medium Sand, trace Clay, some Iron oxide staining, medium dense-moist							
	X	14					22			63		
25												
	X	18					20			63		
30												
					Brown fine to medium Sand, trace Silt, trace fine Gravel, medium dense to dense-moist							
	X	28					12			13		

TBL 05G289.GPJ SOCALGEO.GDT 12/12/05

Southern California Geotechnical

INC

BORING NO. B-2

JOB NO.: 05G289
PROJECT: Proposed MFR Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 12/5/05
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joaquin Baca

WATER DEPTH: 41 feet
CAVE DEPTH: 29 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown fine to medium Sand, trace to little Silt, trace fine Gravel, moist							
					Dark Gray Brown medium to coarse Sandy Gravel, very dense-moist to wet							
	X	87					6			12		
40												
					@ 41 feet, Ground water encountered during drilling							
					Brown Silty fine to coarse Sand, some fine Gravel, some Iron oxide staining, dense-wet							
	X	44					19			47		
45												
					Dark Gray Brown to Black fine Sandy Silt, medium dense-wet							
	X	23					28			72		
50												

Boring Terminated at 50'

TBL 05G289.GPJ SOCALGEO.GDT 12/12/05

TEST BORING LOG

Southern California Geotechnical INC.

BORING NO.
B-3

JOB NO.: 05G289
PROJECT: Proposed MFR Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 12/5/05
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joaquin Baca

WATER DEPTH: Dry
CAVE DEPTH: 23 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
		15			ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt, trace fine root fibers, slightly porous, loose to medium dense-damp to moist	100	13					
		10			@ 3 feet, moderately porous	92	15					
5		12			Brown fine Sandy Silt, trace to little Clay, loose to medium dense-damp to moist	102	17					
		16				106	20					
10		16				104	21					
15		13			Brown Silty fine Sand, trace Clay, medium dense-moist		20					
20		16			Brown fine Sandy Silt, some iron oxide staining, medium dense-moist		21					
25		21			Brown Silty fine Sand, trace Clay, medium dense-moist		18					
					Boring Terminated at 25'							

TBL 05G289.GPJ SOCALGEO.GDT 12/12/05

TEST BORING LOG

PLATE B-3

Southern California Geotechnical
INC.

JOB NO.: 05G289
PROJECT: Proposed MFR Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 12/5/05
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joaquin Baca

WATER DEPTH: Dry
CAVE DEPTH: 16 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt, loose to medium dense-damp							
	X	6					8					
	X	16					8					
5					Brown to Dark Brown fine Sandy Silt, loose to medium dense-damp to moist							
	X	10					13					
	X	9					17					
10												
	X	12					20					
15												
					Dark Gray Brown Silty fine Sand, trace Clay, some fine Gravel, medium dense-moist							
	X	23					22					
20					Boring Terminated at 20'							

TBL 05G289.GPJ SOCALGEO.GDT 12/12/05

TEST BORING LOG

Southern California Geotechnical INC.

BORING NO. B-5

JOB NO.: 05G289
PROJECT: Proposed MFR Development
LOCATION: Lake Elsinore, California

DRILLING DATE: 12/5/05
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joaquin Baca

WATER DEPTH: Dry
CAVE DEPTH: 27 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	BLOW COUNT	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)
		ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt, trace fine root fibers, slightly porous, medium dense-damp		
	20		98	9
	21		98	8
5	17		103	9
		Brown fine Sandy Silt, slightly porous, loose to medium dense-damp		
	13		96	14
10	19		103	11
		Brown fine Sandy Silt to Silty fine Sand, medium dense-moist		
15	13			23
		Brown Silty fine Sand, medium dense-moist		
20	14			20
25	12			26
		Brown Silty fine to medium Sand and some fine Gravel, medium dense-damp		
30	26			4
		Boring Terminated at 30'		

Other columns (empty): SAMPLE, POCKET PEN. (TSF), GRAPHIC LOG, LIQUID LIMIT, PLASTIC LIMIT, PASSING #200 SIEVE (%), UNCONFINED SHEAR (TSF), COMMENTS.

TBL 05G289.GPJ SOCALGEO.GDT 12/12/05

TEST BORING LOG

Southern California Geotechnical INC.

BORING NO. B-6

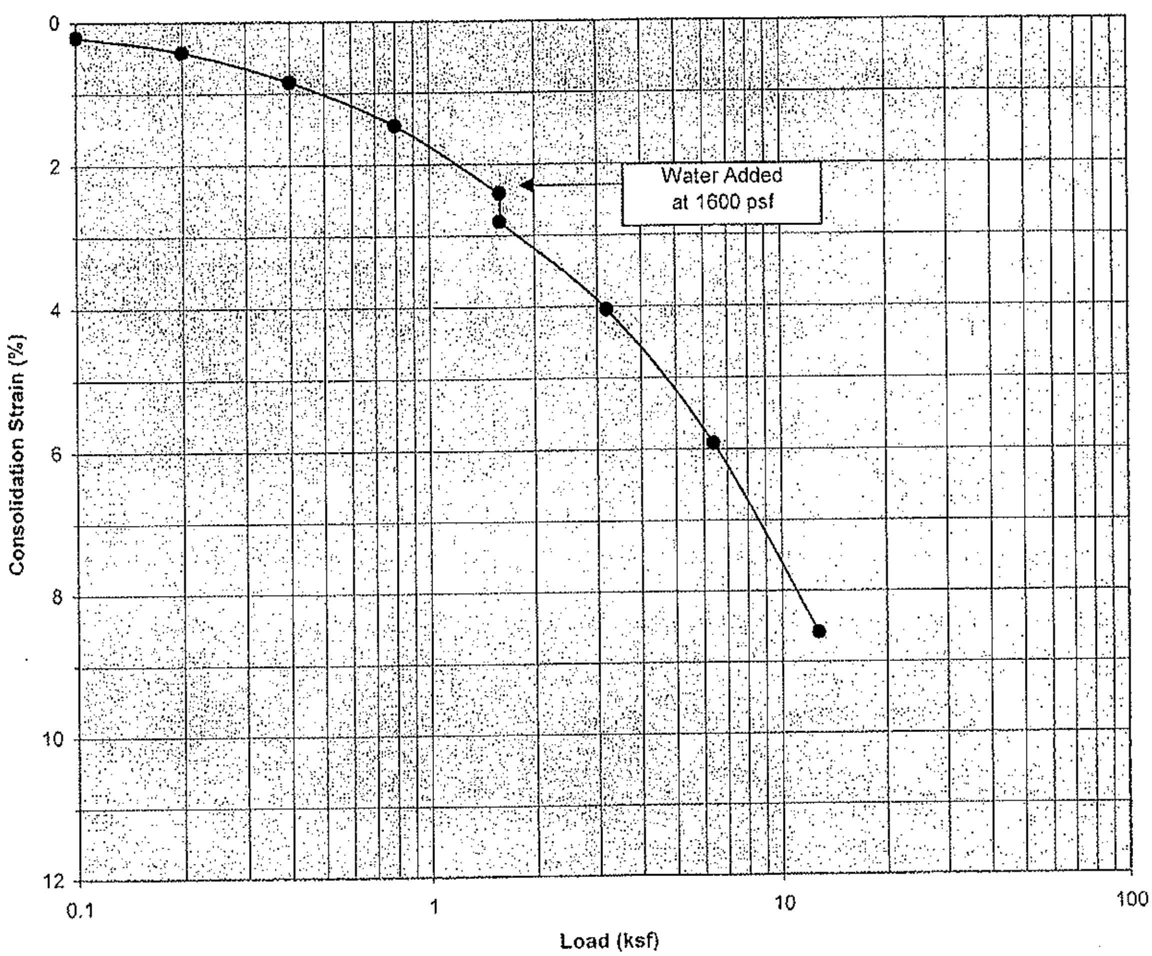
JOB NO.: 05G289	DRILLING DATE: 12/5/05	WATER DEPTH: Dry
PROJECT: Proposed MFR Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 23 feet
LOCATION: Lake Elsinore, California	LOGGED BY: Joaquin Baca	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					<u>ALLUVIUM:</u> Brown fine Sandy Silt to Silty fine Sand, slightly porous, trace calcareous veining, loose to medium dense-damp							
	X	5					8					
	X	20					10					
5												
					Gray Brown fine Sandy Silt, loose-moist							
	X	8					17					
					Dark Brown Silty fine Sand, trace Clay, iron oxide staining, loose-moist							
	X	9					21					
10												
					Brown fine Sandy Silt to Silty fine Sand, medium dense-damp to moist							
	X	13					21					
15												
	X	13					8					
20												
	X	15					17					
25												
					Boring Terminated at 25'							

TBL 05G289.GPJ SOCALGEO.GDT 12/12/05

TEST BORING LOG

Consolidation/Collapse Test Results



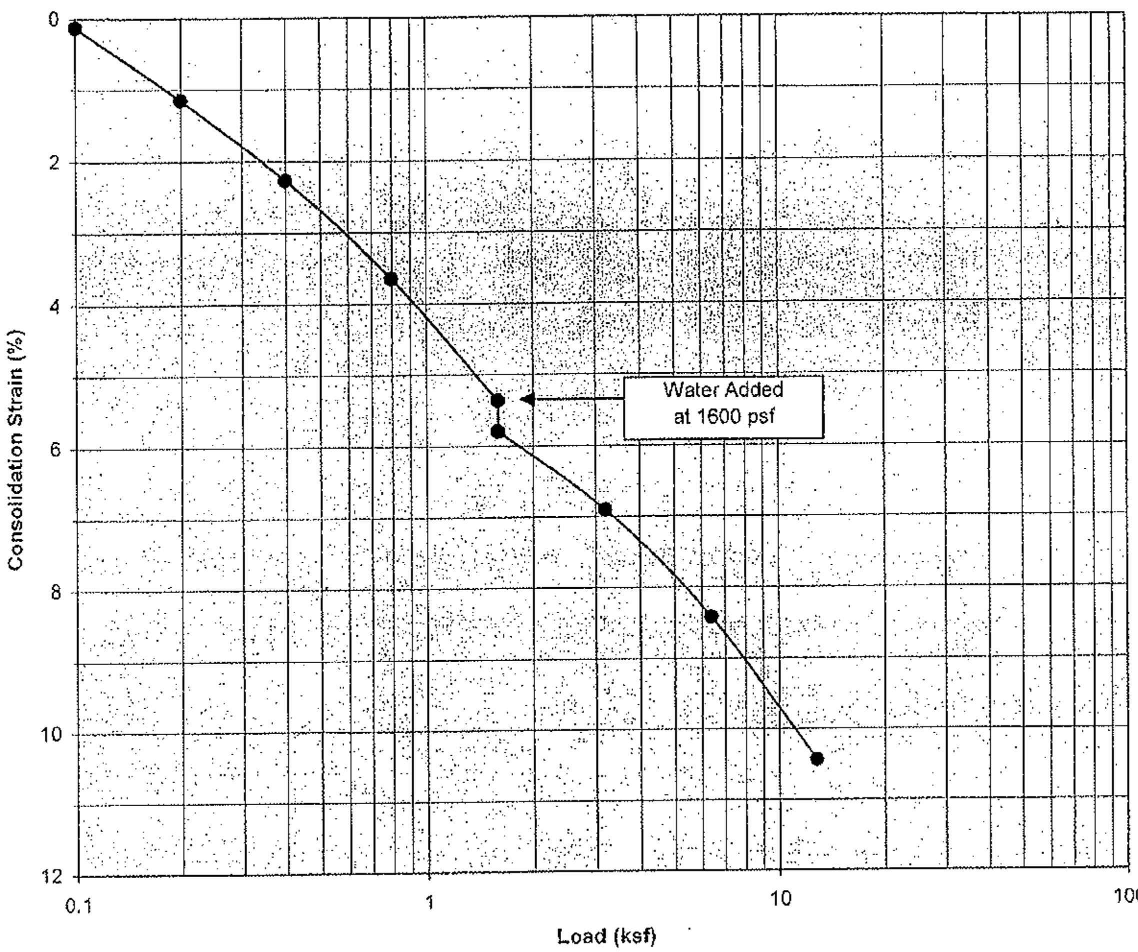
Classification: ALLUVIUM: Dark Brown to Black Silty fine Sand to fine Sandy Silt

Boring Number:	B-1	Initial Moisture Content (%)	25
Sample Number:	---	Final Moisture Content (%)	30
Depth (ft)	1 to 2	Initial Dry Density (pcf)	88.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	97.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.40

Proposed MFR Development
Lake Elsinore, California
Project No. 05G289
PLATE C- 1

Southern California Geotechnical
1260 North Hancock Street, Suite 101
Anaheim, California 92807
Phone: (714) 777-0333 Fax: (714) 777-0398

Consolidation/Collapse Test Results



Classification: Brown to Dark Brown fine Sandy Silt

Boring Number:	B-1	Initial Moisture Content (%)	25
Sample Number:	---	Final Moisture Content (%)	35
Depth (ft)	3 to 4	Initial Dry Density (pcf)	84.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	93.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.43

Proposed MFR Development
Lake Elsinore, California
Project No. 05G289
PLATE C- 2

Southern California Geotechnical
1260 North Hancock Street, Suite 101
Anaheim, California 92807
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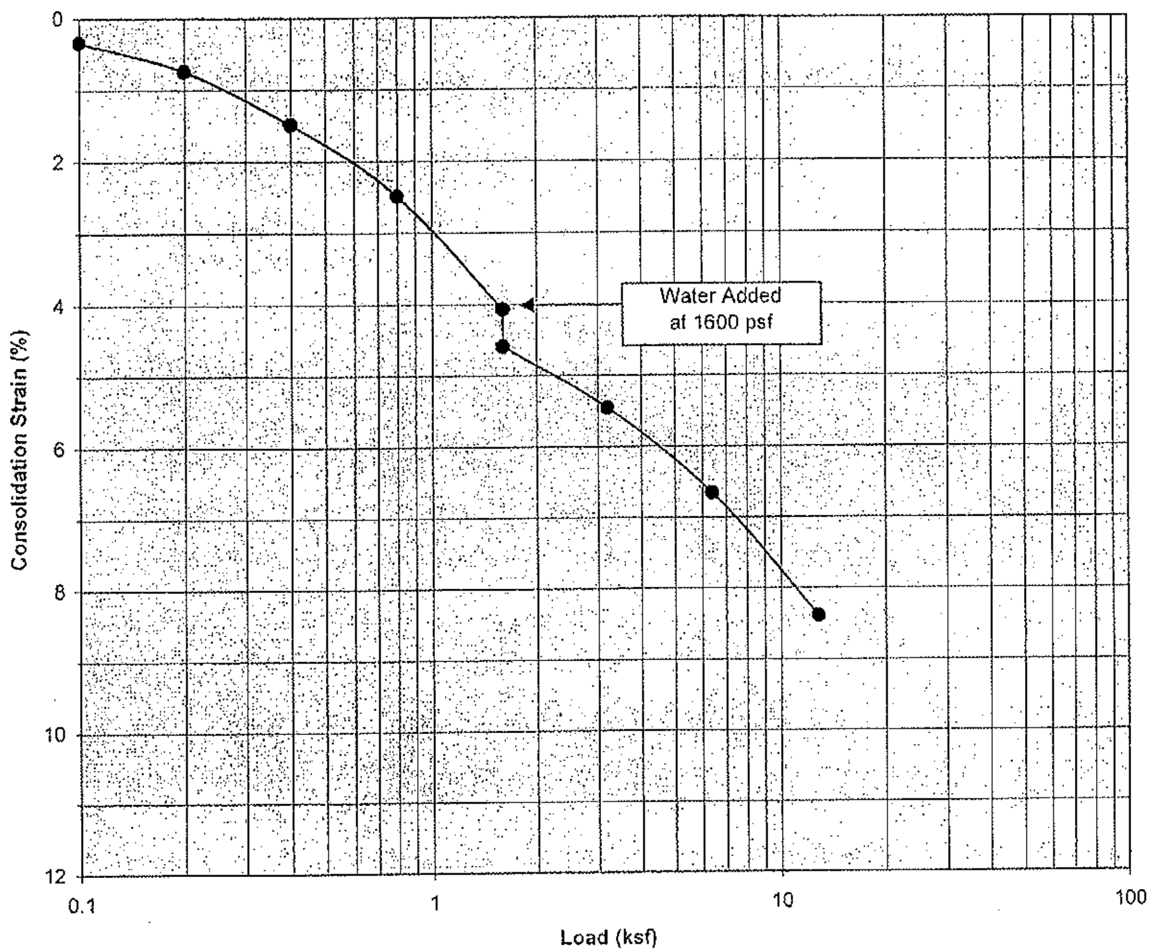
Consolidation/Collapse Test Results



Classification: Brown to Dark Brown fine Sandy Silt

Boring Number:	B-1	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	25
Depth (ft)	5 to 6	Initial Dry Density (pcf)	87.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	94.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.52

Proposed MFR Development
Lake Elsinore, California
Project No. 05G289
PLATE C- 3

Southern California Geotechnical
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Anaheim, California 92807
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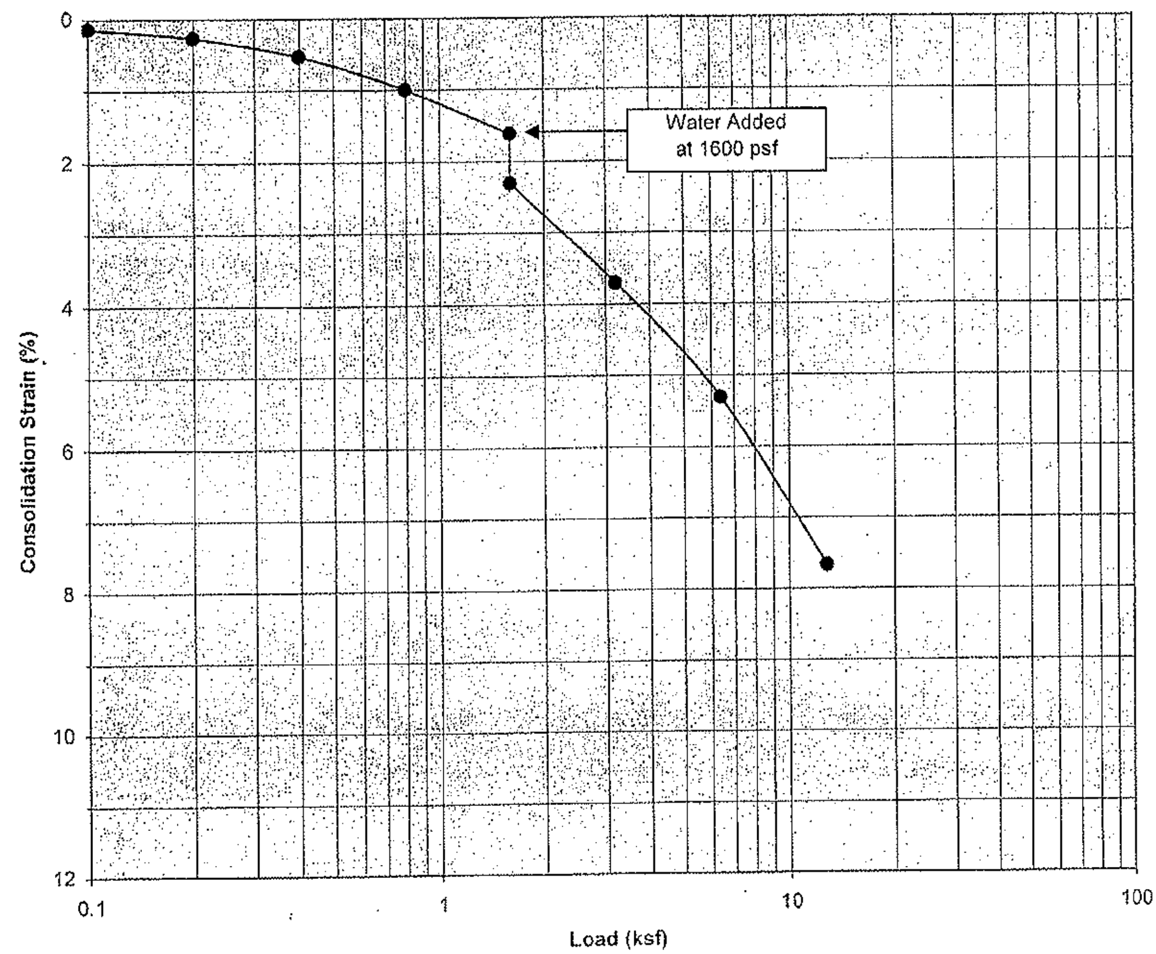
Consolidation/Collapse Test Results



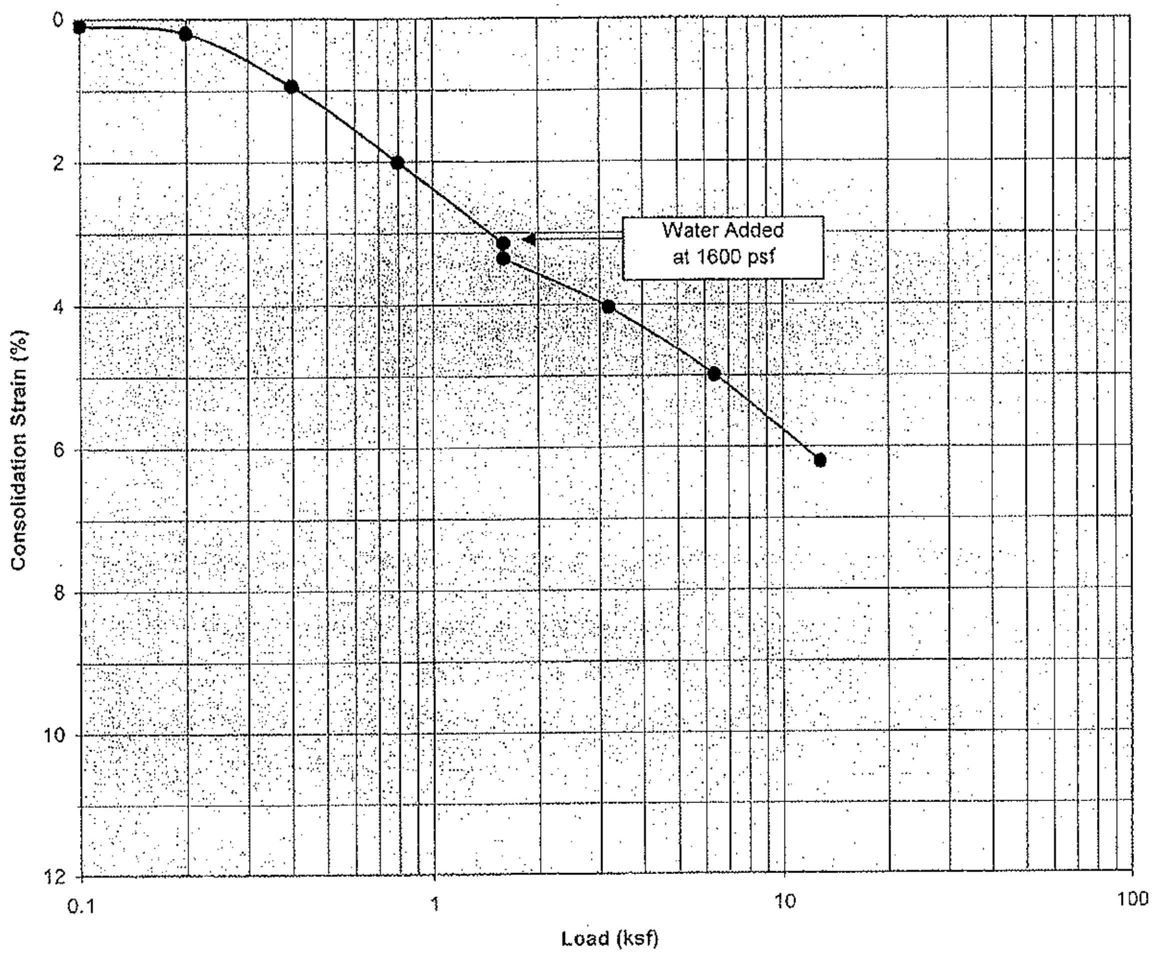
Classification: Dark Brown Silty fine to medium Sand

Boring Number:	B-1	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	7 to 8	Initial Dry Density (pcf)	102.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.69

Proposed MFR Development
Lake Elsinore, California
Project No. 05G289
PLATE C- 4

Southern California Geotechnical
1260 North Hancock Street, Suite 101
Anaheim, California 92807
Phone: (714) 777-0333 Fax: (714) 777-0398

Consolidation/Collapse Test Results



Classification: ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-3	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	1 to 2	Initial Dry Density (pcf)	98.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	105.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.21

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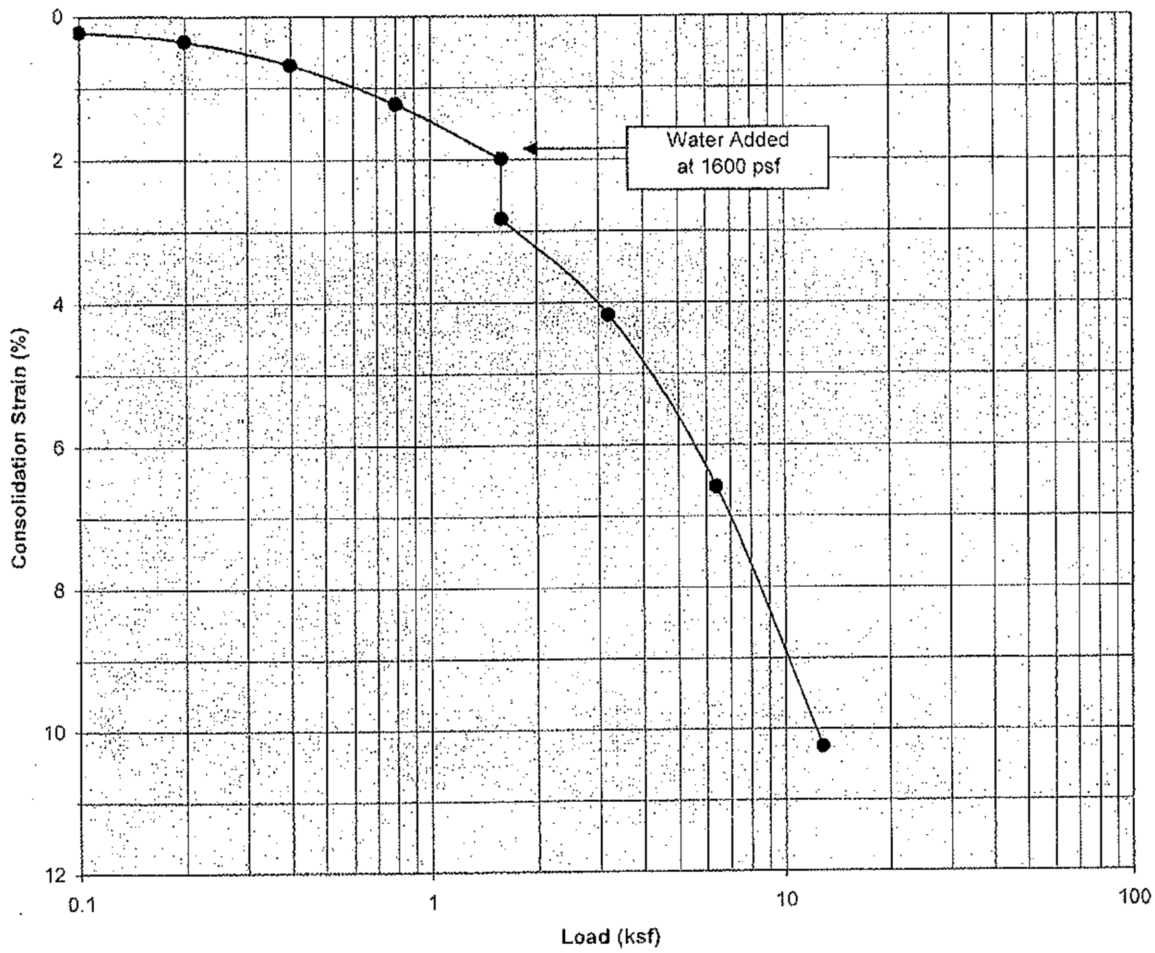
Consolidation/Collapse Test Results



Classification: ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-3	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	3 to 4	Initial Dry Density (pcf)	93.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	104.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.84

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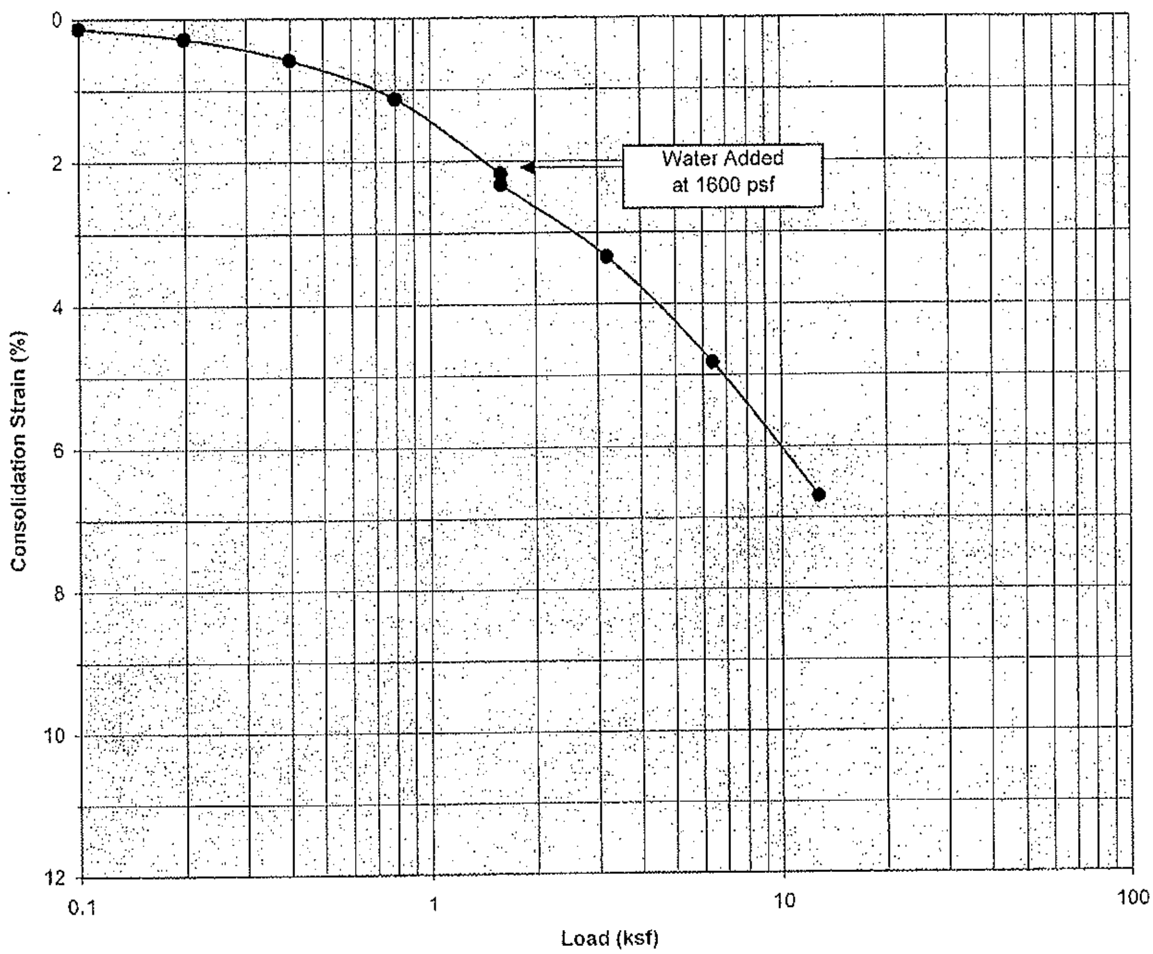
Consolidation/Collapse Test Results



Classification: Brown fine Sandy Silt, trace to little Clay

Boring Number:	B-3	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	5 to 6	Initial Dry Density (pcf)	101.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.15

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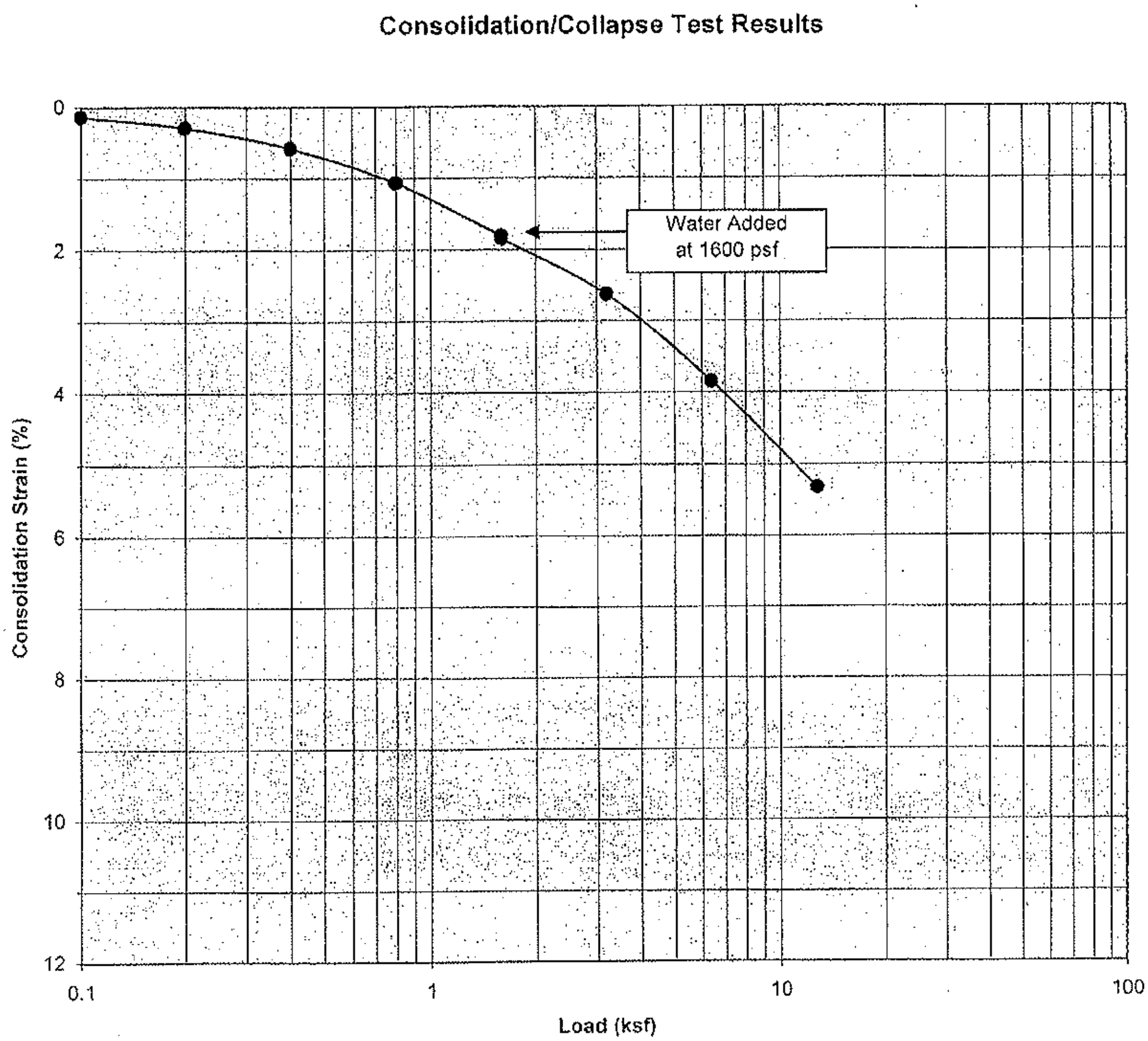

Classification: Brown fine Sandy Silt, trace to little Clay

Boring Number:	B-3	Initial Moisture Content (%)	19
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	7 to 8	Initial Dry Density (pcf)	107.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.04

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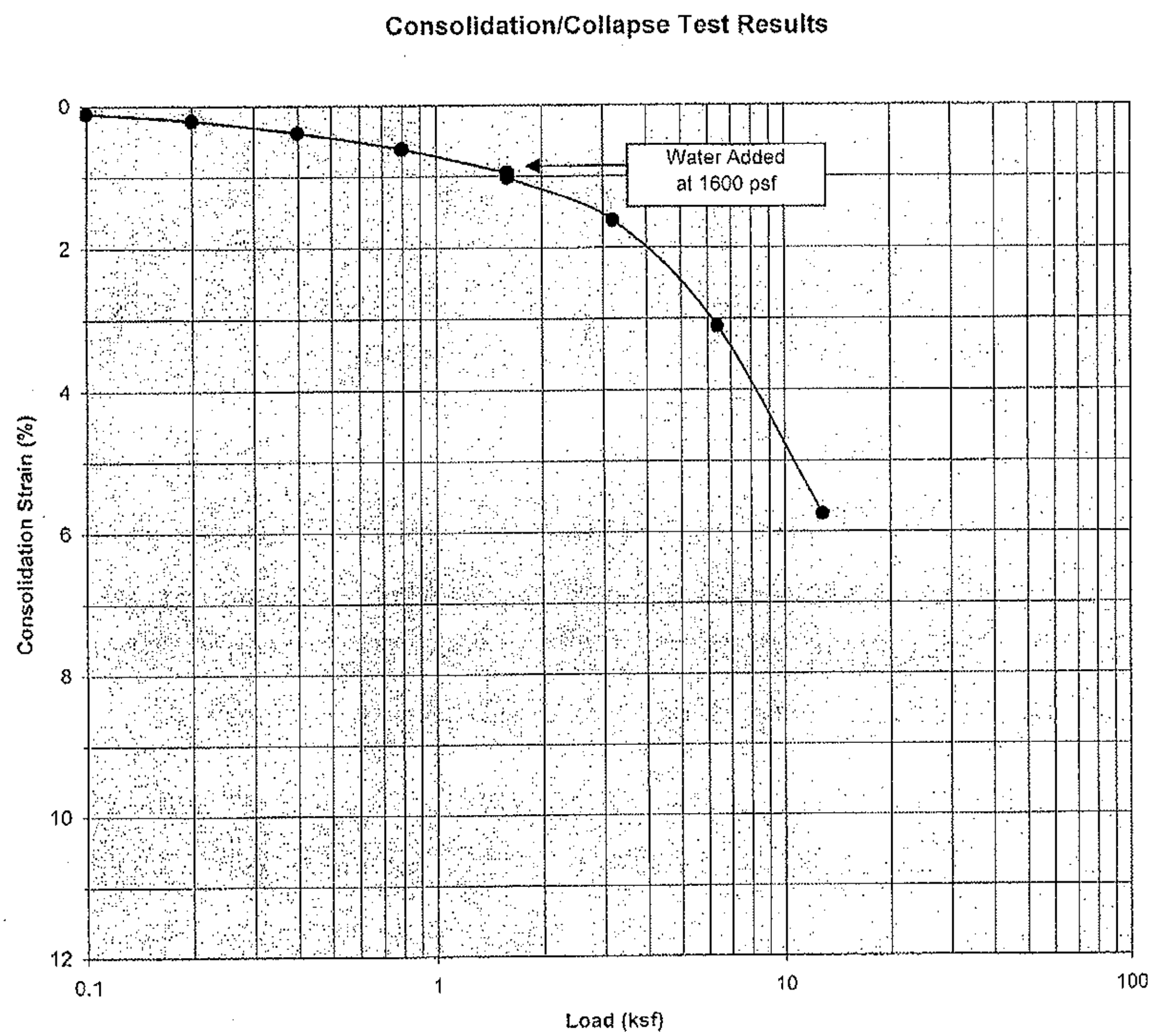

Classification: ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-5	Initial Moisture Content (%)	9
Sample Number:	----	Final Moisture Content (%)	23
Depth (ft)	1 to 2	Initial Dry Density (pcf)	97.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	103.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.07

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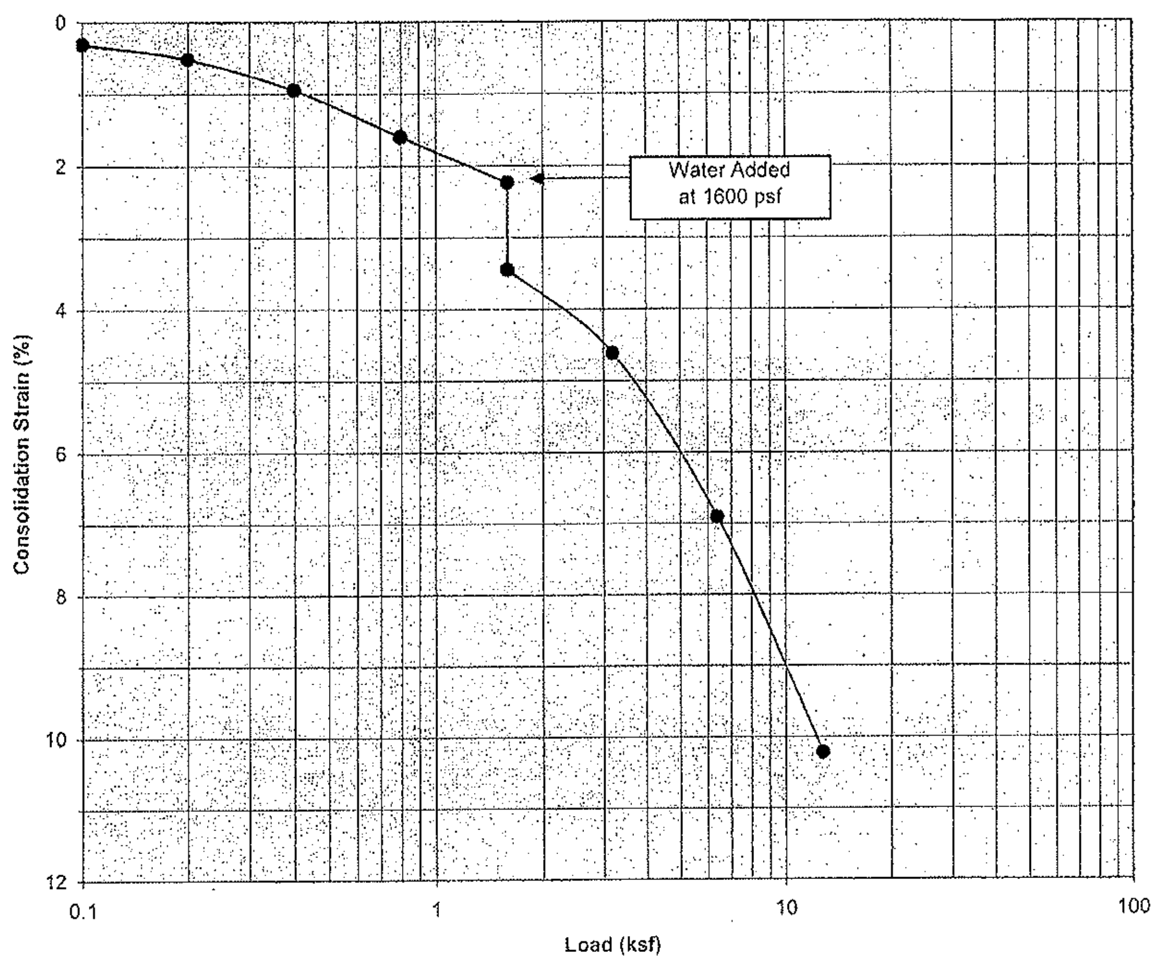
Consolidation/Collapse Test Results



Classification: ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-5	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	3 to 4	Initial Dry Density (pcf)	97.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	107.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.21

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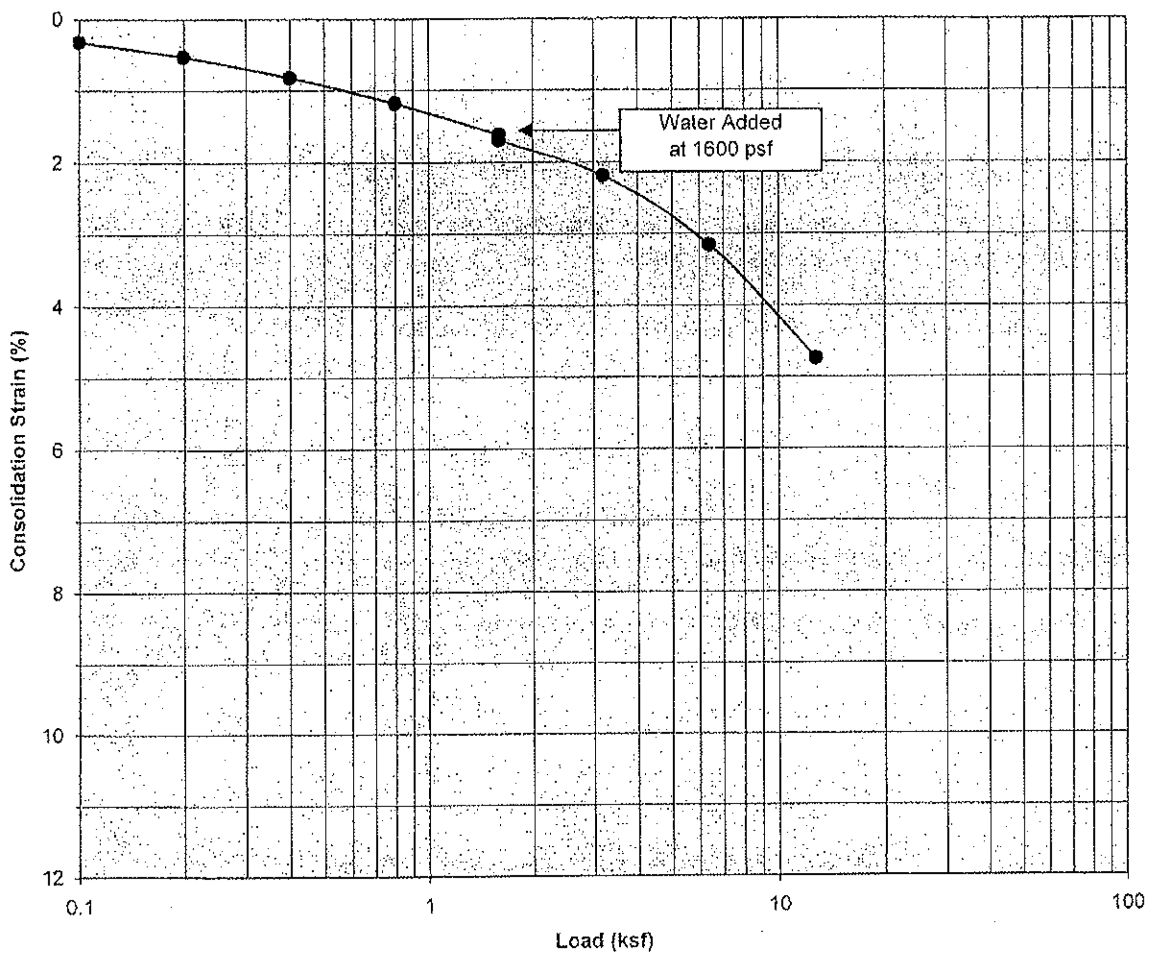
Consolidation/Collapse Test Results



Classification: ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-5	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	5 to 6	Initial Dry Density (pcf)	100.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	106.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.08

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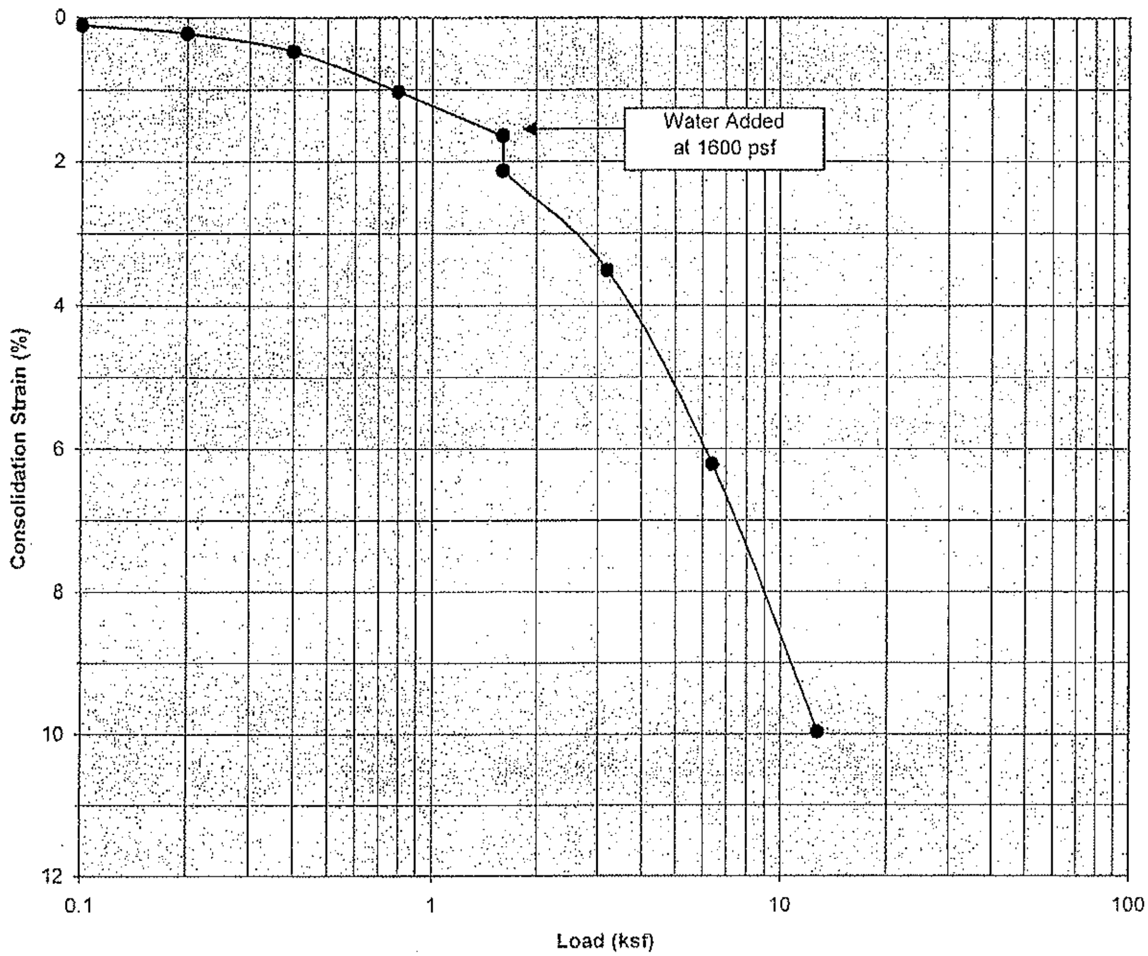
Consolidation/Collapse Test Results



Classification: Brown fine Sandy Silt

Boring Number:	B-5	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	7 to 8	Initial Dry Density (pcf)	95.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	105.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.49

Proposed MFR Development
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